STRUCTURAL DRAFTING

AND

THE DESIGN OF DETAILS

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PREFACE

THIS book has been prepared especially to meet the requirements of engineering students, structural draftsmen, and apprentices in structural drafting. It corresponds in scope to the duties of the structural steel draftsmen, and it therefore covers, not only the preparation of the detailed working drawings for steel structures, but also the design of the details of construction. It is a text-book in Structural Drafting, and it may be used as a text-book in elementary Structural Design. As a reference book for structural draftsmen, it gives practical points as well as theory. A knowledge of the use of drawing instruments is presupposed, but the fundamentals of structural drafting are fully presented. The application of these fundamentals is illustrated by the drawings of many different types of members of steel structures. Exceptionally exhaustive are the chapters on the design of beams and the component parts of plate girders. The tables at the end of the book are sufficiently complete for most student courses, so that no steel manufacturers' handbook need be used. Many of the tables are arranged more conveniently for both students and draftsmen than the tables in the usual handbooks, particularly the tables for I-beams, channels, and angles. A more complete outline of the book is given in Chapter I.

The illustrative drawings have all been prepared by the author, but many of them have been adapted from similar or nearly identical drawings kindly furnished by structural companies, to which due acknow ment is here made. The drawings and the standards of the An Bridge Company have been of exceptional value, and the draw the King Bridge Company, the Hay Foundry and Iron Works, tl ton Bridge Works, the Mount Vernon Bridge Company, the P vania Steel Company, and the Central Railroad of New Jerse been used to advantage. Abstracts from the Specifications of the can Railway Engineering Association have been used freely thro the text and in the tables by permission of the Committee on P tions of that Association.

Grateful acknowledgement is made to Professor J. C. Tracy, E the Department of Civil Engineering at Yale University, and to Pr J. R. Schultz, Head of the Department of English at Allegheny C for their helpful criticisms of the manuscript. Professor Tracy gav thought to the perusal of nine of the most fundamental chapters II, and as a result of his constructive criticisms, these chapters, as the others in the book, have been greatly improved. Professor S has read the whole manuscript and has made many valuable sugg

CARLTON T. BI

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PART I — INTRODUCTORY

CHAPTER I

OUTLINE OF THE BOOK — NOTATION — DEFINITIONS

SYNOPSIS: In this chapter are summarized the arrangement, the notation, and the special features of this book. Brief definitions are given to show the meanings of the engineering terms used in the text.

1. **Scope.** — This book is planned to meet the requirements of engineering students, of apprentices, and of structural draftsmen. In scope it is designed primarily to correspond to the duties of the structural draftsman. It is divided into three parts, emphasis being laid upon Part II — Structural Drafting. This Part II includes the fundamentals of structural drafting, such as the methods of representation, of billing, and of dimensioning. The application of these fundamentals to the drawings of common types of members is fully illustrated. In Part III the elements of design are shown with special reference to the design of details and of simple members with which the draftsman must be familiar. The design of other members and of complete structures is omitted for two reasons, (1) because they are usually designed in a special Designing Department instead of in the Drafting Room, and (2) because nearly all phases of structural design are admirably described in so many books that it would be futile to duplicate them here. Part I is introductory and is inserted primarily to give the student a conception of the relation between the drafting room and the other branches of the steel industry. Tables are inserted at the end of the book. These tables may supplant the handbooks of structural steel companies in some student courses. The tables are arranged to minimize the turning of pages when used either in drawing or in designing. This is accomplished by placing on one page the data usually given on several pages. For instance, by separating standard angles from special angles the properties of each may be shown on a single page. Furthermore, the weights and areas of all angles are grouped with the gages of the angles and the areas of holes, an arrangement which is equally convenient for the draftsman when drawing and when designing. The scope and the use of the tables may be seen best by reference to the tables themselves and to the brief descriptions which follow them.

2. **The general arrangement** of the book is planned for convenient reference. It is believed that the text will be used chiefly for reference by students, apprentices, and draftsmen. The shape of the book is such that it will stay open on the desk without the use of weights or clamps. The size of the pages was selected so that the drawings and the tables could be made reasonably large and clear without the use of intolerable folded inset sheets.

1. The author first planned a chapter which would give an **outline of a course of study.** It was felt that such an outline would simplify the assignment of student work, and that it would be useful to apprentices and others who might use the book without instruction. It might also be of service to the younger instructors when planning to use the text for the first time. However, it was concluded that the large majority of readers would have no use for such a chapter. Each instructor should become familiar with the book before he adopts it as a text, and he should have little difficulty in selecting the proper sequence of paragraphs in the fundamental chapters of Part II. The author does not feel that it is wise for the student to devote much time to a study of the text before he begins to draw. The first drawing, if sufficiently simple, may be started at the first exercise, accompanied by references to the most important paragraphs that bear upon the drawing. This may be followed by other drawings each of which illustrates as many new points as the average student can master. The work should be progressive, but no drawing should involve points beyond the student's ability and he should be expected to make each drawing fundamentally correct. It may be difficult to obtain excellent results at the outset owing to the multiplicity of conventions and practical points which are new to most students; but to allow violations of drafting methods and conventions to stand uncorrected is a source of trouble. Reasonable requirements should be made and enforced, and all mistakes should be corrected by the students as a safeguard against repetition. It is recommended that many of the penciled drawings be left uninked in order to facilitate the making of changes and to enable the students to make more drawings within the allotted time. The author has found it satisfactory to use printed forms for beam work at the beginning of the course, for they not only conform to the usual drafting room practice but they enable the student to concentrate his mind on points other than the arrangement of views and of dimensions lines which he can learn gradually. In order to introduce the fundamentals as early in the course as possible it is expedient to have the students make free-hand drawings at their desks or at the blackboard instead of taking time for careful plotting. As soon as possible the data and suggestions given with each new problem should be reduced to a minimum in order to develop the student's resourcefulness. Frequent tests may be given, and one question on each test may well be devoted to the indication of mistakes on a given drawing, i.e., practice in checking.

2. **Numbering.** — In this book all pages are numbered consecutively. Each figure bears the same number as the page upon which it may be found. This facilitates the finding of any figure to which reference is made. When more than one figure is placed upon the same page they are lettered, thus: Fig. 24 (*a*) and Fig. 24 (*b*) are both on page 24. The paragraphs are numbered on each page independently, beginning with 1. This is done to avoid large paragraph numbers.

3. **Cross references** should be made to both the page and the paragraph. Reference to a page alone is not sufficiently specific, particularly where the paragraphs are short as in the fundamental chapters of Part II; reference to a paragraph without the page often results in an extended search because the paragraphs are of unequal lengths and on some pages no paragraph number is found. In this book the page numbers are followed by the paragraph numbers, while the paragraph symbol (¶) and abbreviation (Par.) are replaced by the colon, thus: Page 29 : 4. Furthermore, a distinction is made between two classes of cross references. Where an additional statement is required to supplement the text, needless repetition may often be avoided by reference to another paragraph. These references are important. Other less important references are given for the convenience of those who do not grasp the full significance without them. Unless a distinction were made it is probable that students would either waste time in looking up points which may be obvious to them, or else they would become careless and fail to look up the more important references. In order to derive the greatest benefit from the cross references, therefore, the following interpretation should be made, viz.:

All references which appear as part of the text or as separate sentences should be consulted, as for example: See page 23 : 2.

All references which appear in parentheses need be consulted only in case additional information is desired, as for example: (page 29 : 4).

4. **Type.** — Bold-faced type is used for convenient reference to show the topic of each paragraph. In some cases separate paragraph headings are used, but this plan is not rigidly adopted. Usually words or phrases

are found which give the desired information, and when these are accentuated it seems unnecessary to duplicate them in special headings. In this manner the more important parts of a paragraph are often emphasized. Fine type is used to insert remarks or explanations at points where they would be more serviceable than in footnotes, but in such a manner that the sequence of the main text remains unbroken.

1. **Formulas** are so often used blindly by persons who have little conception of their meaning that the author sometimes feels that no formulas should be used by students lest they become "handbook engineers." He believes that no one should use a formula who is not positive that he can derive it. In other words a formula should be considered an expression of a method arranged for convenience in applying that method. It is a "short-cut" application of the method but it should not be used without a knowledge of the underlying principles. Comparatively few formulas are used in this book. These are not summarized for reference but appear only in conjunction with the corresponding derivations which in turn are clearly indexed. It is hoped that this plan will tend to cultivate the proper use of formulas.

2. **Notation.** — Considerable annoyance has been experienced on account of the lack of uniformity in the notation commonly used in formulas. For example, students constantly confuse bending moments and resisting moments in pound-feet with those in pound-inches; similarly in the usual reduction formulas, they often substitute the lengths of compression members in feet instead of in inches. While these mistakes are due chiefly to carelessness, yet they would be minimized by greater uniformity. In this book, all dimensions in feet, all moments in pound-feet, and all quantities which involve compound units based upon the foot are expressed by capitals. Similarly, all dimensions in inches, all moments in pound-inches, and all quantities which involve compound units based upon the inch are (with two exceptions) expressed by lower-case letters.

The capital I is used for moments of inertia in inches to the fourth power, and the capital E is used for modulus of elasticity in pounds per square inch. These letters are so universally and almost exclusively used that it seems unwise to institute a change.

Significant letters, or letters commonly used, have been chosen wherever feasible, even though a single letter may thus have more than one meaning. It is felt that these meanings are so distinct that there is little chance for ambiguity; further distinction is made possible by the use of primes or subscript letters. Letters used in the tables at the end of the book or in certain other places are not included in the following summary, since these letters have no special significance outside of the places where their meanings are apparent. The term "pound-feet" is adopted after due deliberation in preference to the more common term "foot-pounds." This is done primarily to distinguish the unit of moments from the unit of work. Both are compound units derived from the product of weights or forces in pounds by distances in feet; but work is measured by the product of a force by the distance through which the body acted upon *moves*, whereas a moment is the product of a force by the perpendicular distance from the force to a point about which there is a *tendency to rotate*. It is confusing, for example, for a student in going from a class in Mechanics to one in Structural Design to find the same term used for these two distinct meanings. Furthermore, it seems more natural to define a moment as "the product of a force by its lever arm" rather than as "the product of the lever arm of a force by the force itself." In practice it is customary first to select a force whose moment is desired, and then to find the distance from this force to the point of moments. Thus it is logical to put the unit of the force before the unit of the lever arm.

NOTATION

= lb. or lbs. = pound or pounds; also number.
′ = ft. = foot or feet.
□′ = sq. ft. = square foot or square feet.
ft. = lb. ft. = pound-feet.
#/ft. = pounds per foot.
#/sq. ft. = pounds per square foot.

ϕ = diam. = diameter.
″ = in. or ins. = inch or inches.
□″ = sq. in. = square inch or square inches.
in. = lb. ins. = pound-inches.
#/in. = pounds per inch.
#/sq. in. = pounds per square inch.

A = area, in square feet.

B = panel length, in feet.

C = compression.; compressive stress, in pounds or in thousands of pounds.

D = degree of curvature, in degrees.; depth, or diameter, in feet.

D_b = depth from back to back of angles, in feet.

D_g = effective depth between centers of gravity, in feet.

D_r = mean depth between rivet lines, in feet.

D_s = depth between centers of splice plates, in feet.

D_w = depth of web plate, in feet.

E* = stress in rivet at unit distance from center of rotation due to eccentricity, in pounds.; modulus of elasticity, in pounds per square inch.

F = flange stress, in pounds or in thousands of pounds.

I* = moment of inertia, in inches^4.; conventional sign for I-beam.

L = left.; length, in feet.; conventional sign for angle.

a = area, in square inches.

b = back.; breadth of beam, in inches.; unit stress in bearing, in pounds per square inch.

c = center.; width of strip, in inches.; distance from neutral axis to the extreme fiber, in inches.

d = depth, or diameter, in inches.

d_b = depth from back to back of angles, in inches.

d_g = effective depth between centers of gravity, in inches.

d_r = mean depth between rivet lines, in inches.

d_s = depth between centers of splice plates, in inches.

d_w = depth of web plate, in inches.

e = eccentricity, in inches.

f = rivet stagger, in inches.; unit stress in the extreme fiber due to bending, in pounds per square inch.

g = gage, in inches.

h = effective depth of arch, in inches.

k = distance in eccentric connections from the center of gravity to the center of rotation, in inches.

l = length, in inches.

* The capitals E and I are used instead of lower-case letters in order to conform to common usage (see preceding paragraph).

M = moment, in pound-feet.
M_B = bending moment, in pound-feet.
M_R = resisting moment, in pound-feet.

P = concentrated load, in pounds or in thousands of pounds.

R = { right.
radius of curvature, in feet.
reaction, in pounds or in thousands of pounds.
R_L = left-hand reaction, in pounds or in thousands of pounds.
R_R = right-hand reaction, in pounds or in thousands of pounds.

S = stress, in pounds or in thousands of pounds.

T = { tension.
tensile stress, in pounds or in thousands of pounds.
conventional sign for Tee.

U = *unit* load uniformly distributed, in pounds per foot or in thousands of pounds per foot.
U' = *unit* load uniformly distributed, in pounds per *square* foot.

V = { velocity, in feet per second.
vertical shear, in pounds or in thousands of pounds.

W = *total* load uniformly distributed, in pounds or in thousands of pounds.

X = { length of cover plates, in feet.
unknown distance, in feet.

Y = unknown distance in feet.

Z = conventional sign for Z-bar.

m = moment, in pound-inches.
m_B = bending moment, in pound-inches.
m_R = resisting moment, in pound-inches.

p = { rivet pitch, in inches.
projection of bearing plate, in inches.

q = statical moment, in inches^3.

r = { radius of gyration, in inches.
radius of curvature, in inches.
limiting value of one rivet, in pounds or in thousands of pounds.

s = { section modulus, in inches^3.
unit stress in shear, in pounds or in thousands of pounds.

t = thickness, in inches.

v = intensity of shear, in pounds per square inch.

x = { unknown distance in inches.
horizontal distance in eccentric connections from a rivet to the center of gravity of a group of rivets, in inches.

y = vertical distance in eccentric connections from a rivet to the center of gravity of a group of rivets, in inches.

z = direct distance in eccentric connections from a rivet to the center of rotation, in inches.

+ = sign for upward forces, for forces toward the right, and for clockwise moments.

− = sign for downward forces, for forces toward the left, and for counterclockwise moments.

C. L. or ℄ = center line.

Sym. abt. ℄ = symmetrical about the center line.

b. to b. = back to back.

c. to c. = center to center.

o. to o. = out to out.

Pl. = conventional sign for plate.

D. L. = dead load.

L.L. = live load.

O.S. = outstanding.

U.M. = universal mill (for plates with rolled edges.)

⊔ = conventional sign for channel.

H equation = $(\Sigma H = 0)$, or the sum of the horizontal components equals zero.
V equation = $(\Sigma V = 0)$, or the sum of the vertical components equals zero.
M equation = $(\Sigma M = 0)$, or the sum of the moments equals zero.

DEFINITIONS

1. On the following pages are summarized brief definitions of the engineering terms used in Parts II and III of this book.* The definitions of some of these terms are not found elsewhere, and the definitions of many others are not readily accessible. It is hoped that the student will be encouraged to consult these pages whenever the meaning of an engineering term is not fully comprehended. No attempt has been made to give every interpretation of the words defined, nor to define such common words as bridge, building, etc. It is intended that every definition is sufficiently complete to explain the use of the word in the text.

Abutment. The masonry support at the end of a bridge or arch.
Anchor. A device for fastening steel work to masonry.
Anchor Bolt. A bolt which fastens steel columns, girders, etc., to masonry.
Anchor-bolt Plan. A drawing which shows the location of anchor bolts.
Angle. A common structural steel shape the cross section of which is in the form of a right angle.
Apex. Panel point.

* For a more complete glossary of engineering terms, see Waddell's "Bridge Engineering," John Wiley and Sons, Inc., New York.

Architects' Scale. A measuring scale graduated for convenience in plotting dimensions in feet, inches, and fractions of inches, as distinguished from an "Engineers' Scale."
Assembling Marks. A system of marks used on the component parts of a member to facilitate assembling in the shop.
Axle Load. The load from a truck, car, or locomotive applied to a structure through the wheels at both ends of the axle, hence twice the corresponding "wheel load."

Back-check. To approve or check the corrections of a checker.
Base Angle. An angle which connects the bottom of a column to the base plate or cast base.
Base Plate. A distributing plate upon which a column rests.
Batten Plate. A plate used to hold the component parts of a member at the proper distance apart. Generally used in conjunction with lattice bars.
Beam. A member which resists flexure or cross bending. Commonly an I-beam or a channel.
Beam Girder. A member composed of two or more I-beams or channels fastened together by bolts with separators or by cover plates.

Bearing. The support upon which a member rests. The resistance to crushing, as offered by a member which bears against another or upon a support, also as offered by a component part of a member to a rivet or a pin.

Bearing Plate. A plate used to distribute the bearing over a greater area, as at the end of a wall-bearing beam.

Bearing Value. The amount of pressure in bearing, either total or per unit of area.

Bed Rock. The solid rock which underlies the looser sand, gravel, etc.

Bending Moment. A term which expresses the measure of the tendency of a beam to bend. It is the sum of the moments of all external forces on one side of the point of moments.

Bent. A vertical frame or truss used to support other members.

Bevel. The slope of a line with reference to another line.

Beveled Washer. A cast washer arranged to compensate for the inclination between a bolt or rod and the member through which it passes.

Bill. A list of material, such as a shop bill, shipping bill, etc. To prepare such a bill. Also to express the size of a component part of a member on the drawing.

Block and Tackle. A set of pulley blocks with ropes used for hoisting.

Block Out. To cut out by means of a rectangular punch.

Blueprint. The form of reproduction of a drawing which is issued from the drafting department. Blueprints are made from tracings by exposing sensitized paper to the light.

Board Measure. Lumber is measured in units of one foot board measure, equal to one-twelfth of a cubic foot, two dimensions being taken in feet and the third in inches. The abbreviation M. B. M. stands for "thousand feet board measure."

Bore. To enlarge a punched hole by means of a cutter which accurately pares the inner surface.

Box Girder. A compound girder with two or more web plates which, with the cover plates, form a closed box.

Box Section. A member in which the component parts enclose a space which is accessible only at the ends.

Brace. An inclined member placed between other members to make a structure more rigid.

Bracing System. A series of diagonals and struts placed between main members to resist wind or other lateral forces.

Bracket. A projecting type of connection usually made of a plate and angles.

Buckle. To bend or bow transversely under the effects of a force.

Buckle Plate. A steel plate which is buckled or dished at regular intervals to increase its resistance to transverse bending. Used in bins and in the floors of highway bridges.

Building Code. A compilation of the building laws and ordinances of a city which relate to building construction.

Butt Joint. A joint in which the ends of the parts connected are cut to bear against each other. The ends are held in place by means of splice plates, or similarly.

Calk. To make the seams of boats, tanks, etc., watertight, either by driving oakum or something similar into the seams, or by forcing the sharp beveled edge of one of two overlapping steel plates into the face of the other by means of an air hammer.

Camber. A comparatively flat vertical curve placed in the bottom chord of a truss or girder to counteract the sag.

Cantilever Beam or **Cantilever Girder.** A beam or girder which projects beyond one or both supports. A cantilever beam may have one end embedded in a wall and the other end unsupported.

Cap Angle or **Cap Plate.** An angle or plate at the top of a column or portion of a column.

Casting. Anything formed by pouring molten iron, steel, or other material into a mold and allowing it to harden.

Center of Gravity. That point through which the resultant of the parallel forces of gravity acting upon a body in any position must pass. If the body could be supported at this single point it would remain in equilibrium in any position.

Center Punch. A cylindrical piece of steel with a sharp point protruding from one end. It is inserted in the holes of templets and struck with a hammer to make dents in the steel to indicate where holes are to be punched.

Change Order or **Change Slip.** An order issued to make changes in the material already ordered from the mills.

Channel. A common structural steel shape the cross section of which is similar to that of an I-beam except that the flanges are on only one side of the web.

Check. To approve the correct portions of a drawing and indicate the mistakes. To verify.

Checker. A person in the drafting room who checks the drawings made by others.

Checkered Plate. A steel plate with raised ribs to prevent slipping. Used for floors, stair treads, etc.

Check Marks. Small v-shaped marks or dots placed over dimensions or other quantities to indicate that they have been checked.

Chip. To cut off projecting parts, as with a pneumatic chisel.

Chord. The main top or bottom member, or line of members, in a truss.

Clearance. A space left between members, or parts of members, to allow for inaccuracies in cutting and to facilitate placing them in position.

Clear Span. The length of span from face to face of abutments.

Clearstory. The raised portion of the roof of a mill building or similar structure, arranged with windows in the vertical sides.

Clevis. A forging used to connect a clevis rod to a plate or angle. The clevis is arranged to screw on the end of the rod, and the plate is inserted between two flattened ends through which a pin is passed.

Clip. A small connection angle.

Collision Strut. An auxiliary member which gives intermediate support to the end post of a bridge.

Column. A long member, usually vertical, which resists compression. It is the principal vertical member in a building.

Column Base. The cast-iron base or pedestal upon which a column stands. Also the base plate and angles riveted to the bottom of a column.

Column Formula. A formula by means of which the allowed unit stress in a column is determined. Its values depend upon the ratio of slenderness of the column.

Column Schedule. A drawing upon which is summarized information regarding the composition and the lengths of different sections of the columns in an office building.

Combination Sheet. A printed form upon which a drawing, a shop bill, and a shipping bill are combined.

Combined Stresses. Stresses due to bending combined in the same member with direct stresses due to tension or compression.

Compass. An instrument for drawing circles.

Component. One of two or more parts into which a force or stress may be resolved. The force or stress is the resultant of its components.

Compression Formula. Same as Column Formula.

Compression Member. A member in which the principal stresses tend to compress or shorten the member.

Concentrated Load. A load which is applied over such a small area or to such a small portion of a member or a structure that in effect it may be considered as a single force.

Concrete. An artificial stone made of cement, broken stone, sand and water which are first mixed together, and then placed in position and allowed to harden.

Connection Angle. An angle used for connecting other parts. Connection angles are often used in pairs.

Connection Plate. A plate used for connecting other parts.

Continuous Beam or **Continuous Girder.** A beam or girder which rests upon more than two supports.

Contra-flexure. A change in the direction of bending in a column or a beam.

Cooper's Conventional Loads. A system of concentrated wheel loads of two conventional locomotives followed by a train, commonly used in finding stresses in railway bridges.

Cope. To cut away part of the flange of a beam to avoid interference.

Cored Holes. A hole in a casting made by means of a core in the mold which prevents the metal from flowing into the space.

Cornice. The rain-tight junction of the overhanging roof and the side walls of a building.

Corrugated Steel. Thin sheets of steel which are stiffened by having corrugations rolled in them. They are used for covering the roofs and sides of mill buildings.

Cotter Pin. A cylindrical pin held in place by a split steel key or "cotter" placed through a hole in the pin.

Counter. An adjustable diagonal placed across one of the panels near the center of a bridge in the opposite direction from the main diagonal tension member in the same panel. Its function is to relieve the main diagonal from stresses which might cause compression under certain positions of the live load.

Countersink. To ream a hole to receive the conical head of a rivet, bolt, or screw so that the end thereof will not project beyond the face of the part connected.

Couple. Two equal parallel forces acting in opposite directions and in different lines. The moment of a couple about any point of moments is the product of one force by the perpendicular distance between the two.

Cover Angle. A splice angle placed inside another angle with both legs in contact

Cover Plate. A plate riveted to the flanges of a girder or compression member to increase the area of cross section.

Crane. A hoisting machine arranged to move heavy loads both vertically and horizontally. An overhead traveling crane is commonly used in mill buildings, being supported by longitudinal girders on opposite sides of the building.

Crane Girder or **Crane Runway Girder.** A girder which supports one of the rails upon which a traveling crane runs. Also a girder of the crane itself.

Crimp. To offset the end of an angle by forging so it can overlap another angle without the use of a filler.

Cross Bracing. Bracing with two intersecting diagonals.

Cross Frame. Vertical transverse cross bracing between girders.

Cross Hatch. To draw sloping shade lines signifying a cross section.

Cross Section. A transverse section. Also a view representing the appearance of a structure or member where cut by an imaginary section plane.

Curved Ruler. A guide along which irregular curved lines may be drawn.

Dap. To notch a timber to fit over another timber.

Data Sheet. A sheet upon which are given the necessary data for the manufacturers of cranes, elevators, etc., that they may make them conform to the building requirements.

Dead Load. The comparatively constant static load on a structure due to its weight, etc., as distinguished from the live or moving load.

Deck Bridge. One in which the principal loads are applied to the top chords.

Deflection. A lateral movement at right angles to the principal axis. Also the linear measure of such movement.

Degree. A measure of curvature in railway work. The degree is the angle at the center of a circular curve subtended by a 100-foot chord.

Depth. The principal vertical distance in a horizontal member or structure, or the corresponding dimension in an inclined member.

Derrick. A hoisting machine so pivoted that a load may be swung horizontally.

Design. To proportion one or more members or parts of a structure to properly fulfill the requirements. Also the act of designing or the results thereof.

Designer. One who designs. The title given to one whose principal duty is to design structures.

Design Sheet. A drawing prepared by the designer showing the principal dimensions of a structure and the sizes of the designed members.

Detail. To make a detailed working drawing. Also a connection or other minor part of a member in contradistinction to the main member.

Detailer. One who details. The title given to a draftsman who makes detailed working drawings.

Develop. In drafting, to represent a bent or curved part as if it were flattened into a plane. In designing, to make a connection fully as strong as the part connected.

Diagram. An outline drawing or sketch in which each member is usually represented by only a single line, as an erection diagram or stress diagram.

Diaphragm. A stiffening plate or similar part placed between the webs of a member, or from one member to another.

Die. A steel form used in forging or cutting any piece.

Dimension. A linear measurement indicated on a drawing upon a dimension line which shows its extent and significance.

Dolly Bar. A tool for holding a rivet in place while the opposite end is being hammered to form the second head.

Door Post. A vertical member in a door frame.

Double Shear. The tendency to shear, or the resistance to shear, in two planes.

Drafting. Making working drawings, usually including the designing of the details.

Draftsman. One who drafts or makes working drawings. The title usually includes those who check the drawings.

Drawing. A representation by means of lines drawn by pencil, pen, etc.

Drill. To make a hole by means of a revolving cutting tool or drill.

Driving Clearance. The distance from a rivet to the nearest projection which might interfere with the use of the machine which drives or upsets the rivet to form the head.

Driving Nut. A nut which is temporarily screwed on the end of a bridge pin to protect it while it is being hammered into position.

Eave Strut. A longitudinal strut between the tops of the columns of a building at the eaves.

Eccentric Connection. A connection in which the line of action of the resultant stress does not pass through the center of the group of connecting rivets.

Eccentricity. The distance from the center of gravity to some other point or center line.

Edge Distance. The perpendicular distance from the center of a rivet or a hole to the edge of the piece which contains it.

Effective Depth. The depth between centers of gravity of the chords, or the depth between the centers of pins.

Effective Length or **Effective Span.** The length of span measured from center to center of end bearings.

Elastic Limit. The maximum unit stress below which the unit deformation is proportional to the unit stress.

Elevation. The vertical distance from a reference surface or datum. Also a drawing or view which represents the projection of a member or structure upon a vertical plane.

End Frame. The steelwork in the end of a building, especially when rafters are used instead of roof trusses.

End Post. The vertical or inclined compression member at the end of a bridge truss.

End Shear. The shear for a section taken near the end of a beam or girder. In a simple beam the section is taken just inside of the resultant reaction, where it is maximum.

Engineers' Scale or **Decimal Scale.** A measuring scale graduated in inches and decimals of an inch, as distinguished from an "Architects' Scale."

Equilibrium. The forces which act upon any body are said to be in equilibrium when they so balance each other that the body has no tendency to move. For the "equations of equilibrium" see page 183 : 2.

Equivalent Load. A load or system of loads which causes the same effect as some other load or system of loads.

Erasing Shield. A shield containing holes of different shapes through which parts of a drawing may be erased without disturbing the adjacent parts.

Erection. The assembling and the connecting of the different members of a structure in their proper positions at the site.

Erection Bolts. Bolts used in erection to hold members in position temporarily until the field rivets are driven.

Erection Diagram. An assembly diagram made to show the interrelation between the members of a structure and to guide the erector in placing them in the proper position.

Erection Mark. An identification mark which aids the erector in properly locating a member. Same as Shipping Mark.

Erection Plan. An election diagram, more strictly applied to horizontal projection rather than to elevations.

Erection Seat. A seat angle riveted to a supporting member to hold a girder or similar member in position until the supporting rivets are driven.

Erector. The person in charge of erection, or collectively, the men who erect a structure.

Erector's List. A list of the field rivets and bolts required to make the necessary field connections in a structure.

Estimate. To compile the quantities, weights, and cost of a structure usually in advance of the construction.

Expansion Bolt. A bolt used for attaching steel work to a masonry wall. The bolt is surrounded by a split sleeve which expands in the masonry as the bolt is tightened.

Expansion Rollers. A group of steel cylinders or segments of cylinders placed under the end of a bridge girder or truss to provide free longitudinal movement on account of temperature changes.

Extension Figure. A dimension which extends beyond another dimension on the same line, as for example to the end of a beam (page 86: 5).

External Force. A force such as a load or a reaction which acts upon a member, as distinguished from an internal force or stress.

Extreme Fiber. The fiber which is farthest from the neutral axis.

Eye Bar. A flat bar of rectangular cross section which is upset at each end to form an enlarged head. A hole is bored in this head for the insertion of a pin.

Fabrication. The shop work required to convert the rolled shapes into complete structural members, or in short, the work done in a structural shop.

Face. To plane or smooth a surface. Also the exterior plane surface of any solid.

Factor of Safety. The ratio of the ultimate strength to the allowed working stress.

Falsework. A temporary trestle used to support a structure during erection or demolition.

Fiber. One of the longitudinal elementary filaments which for convenience are considered to exist in a beam or similar member.

Field. A term used in conjunction with the work done on parts of a structure at or near the site, in contradistinction to work done at the shop.

Field Check. A partial checking of the drawings for a structure to insure the proper connection of the members in the field.

Field Connection. A connection of different members in the field.

Field Rivet. A rivet driven in the field, as distinguished from a shop rivet.

Field Splice. A splice made in the field, in distinction to one made in the shop.

Filler. A plate used to fill a space between two surfaces (page 96: 4).

Fillet. The additional metal which forms the curve at the junction of the flange and the web of a rolled shape (page 26: 1).

Finish. To smooth a surface by planing.

Fitter. A shop workman who assembles the component parts of a member and bolts them in position.

Flange. The wider part of an I-beam or similar shape at the edges of the web. Also the corresponding portion of a girder or column; each flange is usually composed of angles or plates and angles.

Flange Angle. An angle in a flange of a girder or similar member.

Flange Plate. A plate in a flange of a girder or similar member.

Flange Rivet. A rivet which attaches the flange angles to the web plate of a girder.

Flange Splice. A splice in the flange of a girder.

Flange Stress. The stress in the flange of a girder due to bending.

Flat. A plate not over 7 inches wide.

Flexure. Bending. Commonly applied to the bending of a beam.

Floor Beam. A beam in a floor. Also a transverse beam or girder placed at the panel points of a bridge to support the longitudinal stringers.

Floor-beam Reaction. The load upon a floor beam at each line of stringers.

Floor Plan. A plan showing the arrangement of the beams, etc., of a floor.

Floor Plate. A plate of a steel floor, such as used in a furnace building.

Footing. The masonry pier or foundation for a column.

Force. That which tends to change the state of motion of a body. A force is known when its magnitude, direction, line of action, and point of application are known.

Forging. An article formed by being hammered while hot.

Foundation. The masonry which supports a steel structure.

Foundation Plan. A plan which shows the layout of a foundation.

Function. A quantity whose value varies to correspond to every variation in some other quantity.

Gable. The triangular portion of the end of a building between the opposite slopes of the roof.

Gage or Gauge. The distance from the back of the web to a rivet line in the flange of a channel or Z-bar; a similar distance in an angle; the distance between rivet lines in a angle or a flange of another rolled shape. Also the clear distance between the heads of the rails of a track, standard gage being 4.708 ft., or 4′ 8½″.

Gas Pipe. Small wrought-iron pipe — often used in short lengths for separators.

Gin Pole. A guyed pole, nearly vertical, equipped with blocks and tackle, used for lifting loads.

Girder. A compound member usually made of plates and angles designed to resist bending due to transverse loads, as a beam.

Girt. A horizontal member in the side or end of a building used to support the side covering such as corrugated steel.

Government Anchor. A short rod with a V-shaped bend in the center, used to anchor the end of a wall-bearing beam.

Graph. A diagram or chart used in determining values by scaling instead of by algebraic computation.

Grillage. Tiers of beams laid across each other and imbedded in concrete to form the footing for a heavily loaded column.

Grip. The combined thickness of metal connected by rivets, bolts, or pins.

Gross Area. The full area of cross section, in contradistinction to net area.

Grout. A liquid mortar which can be poured to fill small voids or to make a smooth finish.

Guard Rail. Auxiliary steel rails between the service rails, or wooden timbers outside the service rails, for keeping a train on the ties in case of derailment.

Gusset Plate. A connection plate which stiffens a connection, such as a plate which connects several members of a truss or a bracing system.

Hand Hole. A hole made for the insertion of a hand in placing bolts or rivets which would be inaccessible otherwise.

Hanger. A vertical tension member used to support a load.

Heel Plate. A gusset plate at the heel, or main support, of a roof truss.

Hinged Joint or **Hinged Shoe.** A joint or shoe arranged with a pin or roller to permit rotation due to the deflection of a truss.

Hip. The junction of the top chord of a truss with an inclined end post. Also the intersection of two roofs, provided the drainage is away from the intersection, as distinguished from a valley.

Hook Bolt. A bolt with a hook at the head end.

I-beam. A common structural shape the cross section of which is in the form of a letter I.

Impact. The increased effect of live loads when suddenly applied. Impact is usually provided for by adding a certain percentage of the quiescent live load.

Indirect Splice. A splice in which the splice plates or angles are not in direct contact with the part spliced.

Information Sheet. A sheet which may accompany a drawing to impart additional information.

Initial Tension. The tension placed in counters and in diagonals of bracing systems to insure tightness.

Internal Force. A stress within a member.

Itemize. To add the item numbers, etc., to a shop bill.

Joist. A beam which supports wooden flooring.

Knee Brace. A short diagonal brace usually placed between a horizontal member and a vertical member.

Lag Screw. A large wood screw with a square head like a bolt head.

Lap Joint. A joint in which the connected parts overlap each other.

Lateral. Sidewise, or at right angles to the principal axis. Also a diagonal member of a system of lateral bracing.

Lateral Plate. A connection plate or gusset plate in a system of lateral bracing.

Lattice Bar. One of a series of zigzagged or crossed bars riveted to separated component parts of a member to hold them in position.

Latticed Girder. A light parallel-chord truss similar to a plate girder except that the web plate is replaced by web members usually made of one or two angles each.

Laying Out. The marking of the steel from templets or otherwise indicating where holes are to be punched and where special cuts are to be made.

Layout. A preliminary drawing or sketch by means of which distances may be determined by scaling.

Lean-to. A building with a roof which leans against another building or a wall. The roof slopes in one direction only, the higher edge being against the other building.

Left. A member is so marked when made exactly opposite a corresponding member marked "right," the latter being represented on the drawing.

Leg. One of the two flanges or parts of the shape called an angle.

Lever Arm. The perpendicular distance from a force to a point of moments.

Linear. Pertaining to line or to length. A linear dimension is usually one measured parallel to the length of a member.

Lintel. A horizontal beam which supports a wall over an opening.

Live Load. A movable load on a structure.

Load. The weight supported by a structure or part of a structure.

Loop-rod. A rod with a loop at the end through which a pin may be passed.

Louvres. Series of horizontal strips of bent sheet steel arranged along the sides of a monitor to provide ventilation and at the same time to exclude rain or snow.

Lug. A small projecting connection, as a connection angle.

Masonry. A general term for structures made of brick, stone, or concrete.

Masonry Plate. A bearing plate placed on masonry.

Material Order Bill. A list prepared in the drafting room showing the material to be ordered from the rolling mills, or elsewhere.

Member. A part of a structure which is completely assembled in the shop and shipped to the site where it is combined with other members.

Mill. The machine or the plant in which plates and shapes are rolled. Also to plane the end of a member by means of a rotary planer or milling machine.

Mill Building. A steel-framed building with a roof of comparatively large pitch and span, but usually without partitions, intermediate floors, or intermediate bracing.

Milled Joint or **Milled Splice.** A joint or splice in which the connected parts are milled to bear against each other.

Milling Machine. See page 31: 1.

Mitered Joint. A joint in which the angle between the connected parts is bisected by the plane of contact.

Modulus of Elasticity. The constant ratio (within the elastic limit) between the unit stress and the unit strain. For steel of all grades this is between 28 and 30 million pounds per square inch.

Moment. The tendency of a force to cause rotation about a given point. It is measured in compound units as pound-inches or pound-feet and is equal to the product of the force by its lever arm.

Moment of Inertia. A term applied to the sum of the products of the elementary areas of a given cross section by the squares of their distances from a given axis about which the moment of inertia is said to be taken.

Moment Plate. A splice plate designed to transmit the stresses in the web of a plate girder due to bending moment.

Monitor. The raised portion of the roof of a mill building or similar structure, arranged to give additional ventilation or light through the vertical sides.

Multiple Punch. A machine that punches two or more holes at once.

Nailing Strip. A strip of wood bolted to a steel beam or other member, to which strip wooden flooring or sheathing is nailed.

Net Area or **Net Section.** The effective area of metal in a cross section. The rectangular areas of all rivet holes cut by the section are deducted from the gross area of the member or part of member under consideration.

Net Width. The effective width of metal in a plate, the diameters of all holes in a section being deducted from the width of the plate.

Neutral Axis. The intersection of a cross section of a beam or girder and the neutral surface.

Neutral Surface. The part of a beam which is neither shortened nor lengthened when the beam is bent.

Office Building. A steel-framed building with intermediate floors and columns, and a comparatively flat roof.

O. G. Washer. A flat round cast-iron washer commonly used under a bolt head or nut in timber construction. One face is of smaller diameter than the other, a reverse curve or "ogee" curve connecting the two.

Order Bill. A material order bill.

Orthographic Projection. See page 33: 3.

Outlooker. A small angle or similar piece fastened to an end purlin of a building to support the roof which overhangs the gable end.

Overrun. The increase in the actual size of a structural shape above the size indicated on the drawing or order bill.

Oxy-acetylene Flame or **Torch.** An outfit used for cutting steel by burning a narrow slot by means of an intense heat.

Packing. The arrangement of the different members on a pin.

Panel. That part of a truss between adjacent panel points.

Panel Point. The intersection of the working lines of different members of a truss.

Parabola. A curve in which the ordinates vary as the squares of the abscissas, or conversely. For the construct on, see page 260.

Pattern. A wooden model for a casting, used in forming the mold.

Pedestal. A cast-steel or cast-iron stool or support for a bridge girder.

Piece Mark. An assembling mark.

Pier. An intermediate masonry support for a bridge. Also a column footing.

Pilot Nut. A nut which is temporarily screwed on the end of a bridge pin to guide it while it is being driven into position.

Pin. A steel cylinder used for connecting the members of a truss, or similarly.

Pin Plate. A reinforcing plate riveted to a truss member to give greater bearing on a pin.

Pitch. The longitudinal distance between adjacent rivets in the main part of a member. Also the ratio of the center height of a roof truss to the span.

Plan. A drawing which represents the horizontal projection of a structure or part of a structure.

Plane. To smooth to a plane surface.

Plate. A flat piece of rolled steel of rectangular cross section.

Plate Girder. A built beam with a solid web plate to which are riveted two flanges composed of angles or angles and plates.

Pneumatic Chisel. A cutting tool, operated by compressed air, used for cutting off projecting parts.

Pneumatic Reamer. A reaming tool, operated by compressed air used for enlarging holes.

Point of Moments. A point where moments are taken, i.e., from which the lever arms of the forces are measured.

Pony Truss. A bridge truss which is not deep enough to permit the use of overhead bracing between the trusses.

Portal Bracing. The bracing in the plane of the end posts of a bridge.

Post. A comparatively small compression member, usually vertical.

Projection Line. A line drawn at right angles to a dimension line to indicate the extent of the dimension.

Punch. To make a hole as explained on page 29 : 5. Also a punching machine.

Purlin. A horizontal longitudinal member which rests upon the top chords of roof trusses to support the roof.

Radius of Gyration. The distance from an axis of rotation to the center of gyration. Numerically it is equal to the square root of the quotient of the moment of inertia about the same axis divided by the corresponding area.

Rafter. An inclined member parallel to the roof slope which is used either to support the purlins in place of a truss, or, resting upon the purlins, to support the roofing.

Rail Clamp. A small casting used for fastening a crane rail to the flange of a supporting girder.

Ratio of Slenderness. The ratio of the length of a compression member to the least radius of gyration of its cross section.

Reaction. The force on a beam, girder, or truss imparted by the support. It is equal and opposite to the pressure of the beam on the support.

Ream. To enlarge a hole by means of a rotating fluted cutter.

Reduction Formula. Same as column formula.

Reinforced Concrete. Concrete in which steel bars are placed to strengthen it.

Reinforcing Plate. A plate used to strengthen the weaker part of a member to develop the strength of the remaining parts.

Resisting Moment. The moment of the internal forces which resist the bending moment on a beam or girder.

Restrained Beam. A beam which is restrained or "fixed" at a support.

Resultant or Resultant Force. The simplest single force or system of forces which can replace a system of forces and have an equivalent effect.

Reversal of Stress. The changing of stress from tension to compression, or vice versa.

Ridge Strut. A longitudinal strut along the ridge or peak of a roof.

Right. A member is so marked when another member marked "left" is to be made exactly opposite from the same drawing.

Right Section. A section at right angles to the principal axis.

Rivet. A short cylindrical rod of steel with upset heads used to rivet or fasten together component parts of a steel structure. One head is formed before the rivet is put in position, the other afterward.

Rivet Code. The conventional representation of rivets under different conditions.

Riveter. One who rivets or operates a riveting machine. A riveting machine. Also an instrument for drawing small circles to represent rivets.

Rivet Line. A line through the centers of a series of rivets.

Rivet Pitch. The longitudinal distance between adjacent rivets in the main part of a member.

Rivet Spacing. The dimensions which locate the centers of rivets.

Rocker. A hinged shoe with a pin or other device to prevent unequal distribution of pressure upon the masonry when the supported girder or truss deflects.

Rod. A rolled bar of steel with round or square cross section.

Roller. A steel cylinder or segment of a cylinder placed under one end of a bridge girder or truss to facilitate longitudinal movement on account of temperature changes. Groups of rollers are held in place by a roller box, the whole forming a roller nest.

Rolling Mill. The machine or the plant in which plates and shapes are rolled.

Rotary Planer. See page 31 : 1.

Rough Bolt. An ordinary bolt, as distinguished from a turned bolt or machine bolt.

Round. A round rod.

Ruling Pen. An instrument for drawing ink lines.

Safe Load. A load which can be supported by a member without overstressing the member. More commonly the maximum safe load.

Sag Rod. A vertical or inclined tie rod used to prevent a girt or a purlin from sagging.

Saw-tooth Roof. See page 113 : 2.

Scale. A flat or triangular measuring stick used in plotting a drawing in proportion to the thing represented Also this proportion.

Seat Angle. A small angle riveted to one member to support the end of a beam or girder.

Secondary Stress. An indirect stress which results because the ideal conditions upon which the calculation of the principal or primary stresses is based are not realized.

Section. A cut across a member or structure made by an imaginary plane. Also used in place of "sectional view."

Sectional View. The projection of one segment of a member or structure upon a section plane.

Section Line. To shade a sectional view by means of fine sloping lines representing the parts cut by a section plane.

Section Modulus. The quotient of the moment of inertia of a cross section of a member by the distance from the neutral axis to the extreme fiber.

Section Plane. An imaginary plane which cuts a section.

Selvage Edge. The original woven edge of a piece of cloth where the threads are closer together than in the body of the cloth.

Separator. A casting or piece of gas pipe placed between the webs of beams to keep them a fixed distance apart.

Shank. The cylindrical part of a rivet or bolt, as distinguished from the head.

Shape. A general term for rolled steel of any cross section other than a plate.

Shear. To cut by shearing (page 28: 1). Also an expression for the algebraic sum of certain forces which tend to shear a member.

Sheared Plate. A plate which is rolled between two rolls and then sheared to the desired width at the mill, as distinguished from a Universal Mill plate which is rolled to the desired width by means of supplementary rolls.

Shearing Stress. The internal forces which resist the tendency to shear.

Shearing Value. The strength of a rivet, pin, or bolt in resisting shear.

Shear Intensity. The shearing stress per unit area.

Shears. A machine for shearing.

Sheathing. A wooden covering of planks or boards.

Shipping Bill. A list of members to be shipped from the shop to the site.

Shipping Mark. An identification mark assigned to each separate member shipped.

Shoe. The part of a bridge that transmits the load from the end pin of a truss to the bearing plate or rollers.

Shop. The place where the component parts of a structure are fabricated into members.

Shop Bill. A summary of material required for fabricating members in the shop.

Shop Drawing. A working drawing prepared for use in the shop.

Shop Rivet. A rivet which is driven in the shop, as distinguished from a field rivet driven at the site.

Sidewalk Bracket. A bracket which supports the sidewalk of a bridge.

Simple Beam. An unrestrained beam which is supported at both ends only.

Single Punch. To punch one hole at a time.

Single Shear. The tendency to shear, or the resistance to shear, in one plane.

Site. The final location of a structure.

Sketch Plate. An irregular plate which is cut to dimension at the mill according to a sketch.

Sketch Sheet. A small sheet or printed form upon which a drawing is made.

Skewback. An auxiliary angle or other support for a floor arch. Also a bent plate or casting used to attach a diagonal rod.

Skew Bridge or **Skew Span.** A bridge or span which does not cross a stream or roadway at right angles; the end of one truss or girder is not opposite the end of the other truss or girder.

Skew Portal. The portal or the portal bracing of a skew bridge.

Skids. Parallel supports of timber or metal used to elevate members a convenient distance above the floor of the shop to make them more accessible.

Slab. A flat solid of considerable area but of relatively small thickness, such as a portion of a concrete floor between the supporting beams.

Sleeve Nut. A long tubular nut having a right-handed thread in one half and a left-handed thread in the other, used for joining two rods and pulling them together to tighten them.

Slope. The bevel or inclination of one line with reference to another; it is measured by the tangent of the angle of inclination expressed in inches to a base of one foot.

Slotted Hole. An elongated hole with semi-circular ends and parallel sides.

Sole Plate. A plate riveted to the bottom of a plate girder to bear upon a masonry plate.

Solid Floor. Any type of floor construction other than the so-called open floor of a railway bridge in which the ties rest directly upon the stringers or girders.

Span. The distance between the supports of a beam, girder, truss, etc. Also a bridge or similar structure which spans an opening.

Specifications. That part of a contract which prescribes the allowed unit stresses and gives directions and restrictions regarding proper construction.

Spiking Piece. A wooden strip bolted to a steel beam or similar member to which strip planking or sheathing may be spiked.

Splice. The connection of two similar members or segments of members in the same straight line.

Splice Angle or **Splice Plate.** An angle or plate used in a splice.

Staggered Rivets. Rivets which alternate on two parallel rivet lines.

Statical Moment. The product of an area by the distance from an axis to the center of gravity of the area (see page 202 : 1).

Statics. That branch of Mechanics which has to do with systems of balanced forces acting upon bodies at rest.

Steel. A modified form of iron used in construction. See Chapter III.

Stiffener or **Stiffening Angle.** An angle used to prevent a plate from buckling or to prevent a seat angle from bending.

Stitch Rivets. Rivets placed at comparatively long intervals, usually in a member composed of two angles, to hold the component parts together and to equalize the stress between them.

Straight-edge. A thin strip of wood, metal, or celluloid with a straight edge used as a guide in drawing straight lines.

Strain. The deformation in a member caused by an external force. Strain is measured in linear units.

Stress. An internal force which resists the tendency of an external force to change the shape of the body.

Stress Diagram. A diagram by means of which stresses are determined graphically. Also a stress sheet.

Stress Sheet. A sheet upon which is recorded the stresses in the principal members of a structure.

Stringers. The longitudinal members which support the track or the floor of a bridge. They are supported by transverse floor beams.

Structural Company. A company engaged in the construction of steel structures.

Structural Drafting. The preparation of the working drawings for the members of a steel structure, such as a bridge, a building, a tower, etc.

Structural Shop. A shop where the rolled steel shapes are punched, cut, riveted, and otherwise prepared for use in a steel structure.

Strut. A comparatively light compression member, usually with no intermediate connection.

Sub-punch. To punch to a smaller diameter.

Substructure. The masonry abutments, piers, or foundation for a steel structure.

Super-elevation. The vertical distance between the tops of the rails of a track on a curve.

Superstructure. The main part of a structure above the masonry foundation or "substructure."

Sway Bracing. Bracing in a vertical plane, as between the columns of a building or between the trusses of a bridge.

Swedge Bolt. An anchor bolt with a nut at one end but with elliptical depressions near the other end to furnish greater bond when imbedded in masonry.

Tamp. To compact concrete, dirt, or other material by pounding.

Tee or T-iron. A structural shape, the cross section of which is in the form of a letter T.

Templet. A strip of wood upon which holes, cuts, etc., are laid out and from which the steel is marked accordingly.

Templet Maker. One who makes templets.

Templet Shop. The shop where templets are made.

Tension Member. A member in which the principal stresses tend to lengthen the member.

Through Bridge. One in which the principal loads are applied to a floor system near the bottom, and the trains, etc., pass "through" the structure between the trusses or girders.

Tie. A light tension member, such as the diagonal in a bracing system. Also a transverse timber which supports the rails of a track.

Tie Plate. A plate used to hold the component parts of a member at the proper distance apart. Generally used in tension members or else in conjunction with lattice bars.

Tier. A row or layer placed above or below a similar row or layer.

Tie Rod. A short rod used to tie the beams of a floor together in order to counteract the thrust from floor arches. Also a rod used similarly elsewhere.

Trace. To copy a drawing or portion of a drawing upon a superimposed transparent sheet of tracing cloth or paper.

Tracer. One who traces. A title given to a person in a drafting room whose chief duty is to trace drawings made by others.

Tracing. A drawing on tracing cloth.

Tracing Cloth. A linen cloth specially treated to make it transparent for use in copying drawings by tracing and blueprinting.

Track. The rails, including their supports, along which a body or structure with wheels or rollers may be rolled. The track on a railway bridge includes not only the service rails and the ties, but also the guard rails, and the bolts, spikes, and other fastenings.

Traveler. A form of derrick used in erection; it is mounted on wheels so that it may be advanced as the work progresses.

Triangle. A flat piece of celluloid or similar material used in drafting. The three edges form a right triangle, and the complementary angles are usually 45°, or else 30° and 60°.

Truss. A framed structure which acts as a beam The principal members form a series of triangles, and each member is primarily subjected to axial stress only.

T-square. A T-shaped drawing instrument with a long thin blade attached to a shorter thicker head. The blade is used as a straightedge for drawing parallel lines as the head is moved along the end of the drawing board.

Turnbuckle. Similar to a sleeve nut except that a transverse opening is provided at the center for the insertion of a crow-bar by means of which the turnbuckle may be turned. See sleeve nut.

Turned Bolt. A machine bolt which is cut in a lathe to accurately fit a hole.

U-bolt. A rod bent in the shape of the letter U with nuts on each end.

Underrun. The decrease in the actual size of a structural shape below the size indicated on the drawing or order bill.

Uniform Load or **Uniformly Distributed Load.** A load which is uniformly distributed over a given distance.

Unit Stress. The stress per unit of area, or the intensity of stress.

Universal Mill Plate or **U. M. Plate.** A plate rolled in a Universal Mill which is provided with vertical rolls as well as horizontal rolls. A plate with rolled edges, as distinguished from a sheared plate.

Upset. To enlarge the end of a rivet, a rod, an eye bar, etc., by hammering or pressing into a die while hot.

Valley. The intersection of two roofs provided the drainage is toward the intersection, as distinguished from a hip.

Vertical Flange Plate. A vertical plate in the flange of a plate girder, either between the web and a flange angle or outside the vertical leg of an angle.

View. In orthographic projection, a view is the projection of an object upon a plane by means of parallel lines.

Washer. Usually a flat disc with a central hole, used under the head or the nut of a bolt, or similarly.

Web. The web plate of a girder, column, or other built member, or the corresponding thin portion between the flanges of an I-beam, channel, etc.

Web Member. An intermediate member of a truss or latticed girder between the chords.

Web Plate. The main plate of a plate girder, column or similar member, connecting the two flanges.

Web Splice. A splice in a web plate.

Wheel Load. The load from a truck, car, or locomotive applied to a structure through a wheel.

Wind Bracing. A system of bracing which resists stresses induced by the wind.

Wind Load. A load on a structure due to wind pressure.

Working Line. A reference line to which the dimensions of a member are referred; usually used in conjunction with the working lines of other members to form a system of working lines of a truss, latticed girder, of bracing system.

Working Point. The intersection of two or more working lines.

Z-bar. A structural shape the cross section of which is in the form of the letter Z.

CHAPTER II

THE ORGANIZATION OF A STRUCTURAL COMPANY — THE ENGINEERING DEPARTMENT

Synopsis: The student should have a conception of the relation which the structural steel drafting department bears to other branches of the steel industry. An abstract is given in Chapters II, III, and IV.

1. **The structural draftsman** is not concerned directly with the manufacture of steel or even with the rolling of the commercial steel "shapes." His drawings show how these shapes are cut, punched, and assembled to form members which in turn go to make steel structures. But every draftsman should understand the processes which are allied to the work of his company. The student has no time for a careful study of the different operations, but he should have a general idea of how steel is made and used. For his convenience an abstract is presented in this chapter and in the two subsequent chapters. Later he may acquire further knowledge from books or from inspection trips to rolling mills, to structural shops, and to erection sites.

2. In the **Estimating or Designing Department** of a structural company are made the preliminary design and the estimate of cost of a proposed structure. These may be based upon the customer's layout, or upon an original design submitted to the customer for approval. Usually several different companies furnish estimates in competition. After a contract is awarded the design sheet is forwarded to the Detailing or Drafting Department. The design sheet usually shows the main form and dimensions of the structure, the principal stresses, and the sizes of all main members, together with special instructions regarding the details. The work of the Designing Department is explained more fully on page 20: 2.

3. In the **Drafting Department** the detailed working drawings are prepared for use in the shop. Various diagrams and lists are also made, such as the preliminary bills of material from which the steel shapes are ordered from the rolling mills. As far as possible, the material must be ordered before the drawings are made so that the mills may roll the steel while the drawings and the templets are being prepared. As soon as the drawings are made and checked, blueprints are sent to the templet shop, to the structural shop, and to others concerned. The work of the Drafting Department is described more fully on page 20: 4.

4. The drawings are first sent to the **templet shop** where templets are made for most of the members. These templets are virtually patterns for cutting and punching the component pieces. They are usually made of wood. Not only can the work be laid out on wood with greater facility than on steel but the templets can often be completed before the steel arrives from the rolling mills and thus the completion of the structure is hastened. Furthermore, the work may be laid out on wood once and then the templets may be used repeatedly in marking many steel pieces which are alike or similar.

5. The **manufacture of steel** and the rolling of the structural shapes are described in the next chapter. The finished shapes are shipped to the **structural shop** where all cuts and holes are first indicated on the steel by means of the templets. The steel is then taken to shears to be cut and to punches to have the rivet holes punched. The component parts of each member are then assembled, being held together temporarily by bolts until the shop rivets are driven. Other proc-

esses may be required on some members before thay are painted and shipped, as described more fully in Chapter IV, page 27.

1. **Erection.*** — The different members of a structure are shipped to the site as far as practicable in the proper sequence for erection. The methods of erecting them differ with the size and the type of the structure and with its location. Usually buildings are made self-supporting from the first, but truss bridges must be supported by "false work" or by other means until they are nearly complete. Locomotive cranes are used extensively in the erection of mill buildings, girder bridges, and viaducts. Derricks are used for office buildings and "travelers" for truss bridges. Main members are usually placed in position first and secondary members are filled in afterwards. Enough erection bolts are used to hold the members in position until the "riveting gangs" can drive permanent rivets.

THE ENGINEERING DEPARTMENT †

2. The **Engineering Department** includes the Designing or Estimating Department and the Drafting or Detailing Department. Both departments are in charge of a Chief Engineer and often one or more Assistant Chief Engineers, although these officers are usually more directly concerned with the work of the Designing Department. The organization of the **Designing Department** differs in different companies and the procedure depends upon the organization and also upon the nature and the magnitude of the proposed structures. Some companies have a special contracting Department which acts as intermediary between the Designing Department and the customers. Some designs are made by the customer's engineers or by consulting engineers, and the structural companies simply estimate the cost and submit bids. Some structures are so simple or so similar to other structures that the designers, or the contracting engineers in charge of branch offices, can make quite accurate estimates quickly without complete designs. Often-times the customer has little conception of the type of structure best suited to his needs and the structural companies prepare alternate designs from which the customer may make selection.

3. **Design sheets** or stress sheets are made by the designer or by a draftsman under his direction to illustrate the proposed structure. They show the general form of the structure, the principal dimensions and stresses, and the composition of each main member, as illustrated in Fig. 21. The design is made according to specifications approved by the customer. An estimator must have an intimate knowledge of drafting room methods and of shop methods and costs. He must know from experience how much to allow for the details of construction such as connection plates and angles. He must be familiar with the methods of erection and be able to determine the method best adapted to a given structure in a given location. The estimator may be assisted by draftsmen, tracers, and computers. As soon as a contract is awarded, the design sheet is adapted to the needs of the Drafting Department and any necessary alterations or additions are made. The design sheet should give all information necessary to enable the draftsmen to make the detailed drawings. It may be supplemented by an "information sheet" which gives the principal terms of the contract such as the time of delivery and whether the cost is quoted at a "lump sum" or at a price per pound.

4. The **Drafting Department** is in charge of a Chief Draftsman. His subordinate draftsmen are usually divided into squads, each in charge of a "squad foreman" or "squad boss." The drawings for each contract are usually all made in one squad, the drawings for other contracts often being carried on simultaneously. The size of each squad varies with the amount of work being done under the direction of a single squad foreman from 3 or 4 to 16 or 20, the normal number being from 6 to 8. In each squad are checkers, detailers, tracers, billers, and computers. The detailers make the working drawings, and design the details. The drawings show how the standard shapes are cut and combined to form the different members. The number and the spacing of the rivets are given so that the members may be properly constructed and so that they may be easily connected to other members in the structure. As far as practical the parts are combined in the shop in

* See Thayer's "Structural Design," Vol. I, D. Van Nostrand Co., New York, and Merriman and Jacoby's "Roofs and Bridges," Vol. III, John Wiley and Sons, Inc., New York. For erection tools and specifications see Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co. Inc., New York.

† See also Tyrrell's "Mill Buildings," The McGraw-Hill Book Co. Inc., New York.

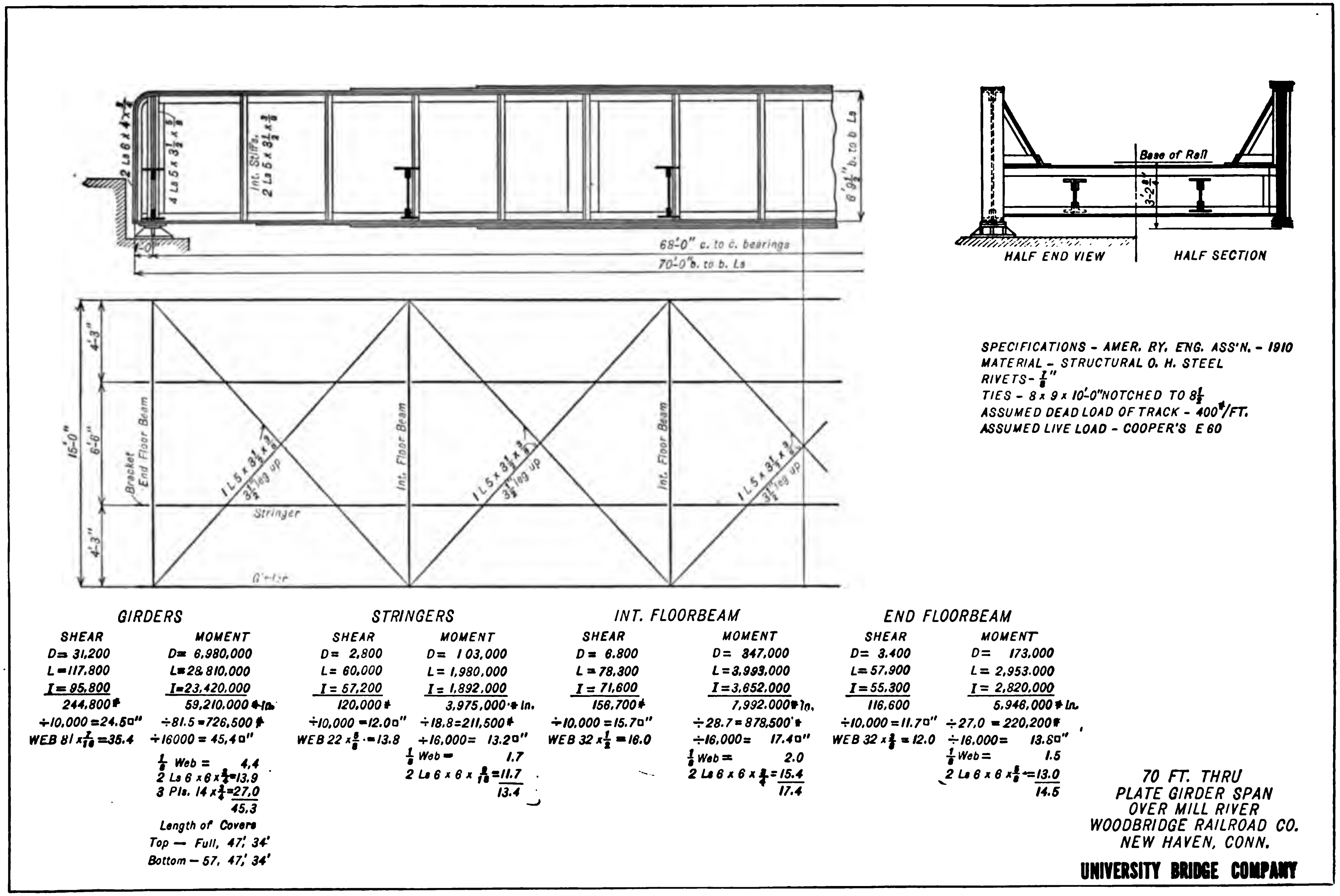

Fig. 21. Typical Design Sheet.

order to reduce the number of members to be shipped and to be erected at the site. The detailers often prefer to trace their own drawings or else to draw with ink directly upon the tracing cloth, thus making no use of the tracers. The detailers are often called upon either to make or to check shop bills and shipping bills and other lists of material. Most of the billing and the calculating of weights are done by younger men of limited experience often working in a separate squad under a chief biller. The checkers "check" or verify the drawings and indicate the mistakes. They are often called upon to make drawings also. They are men of greater experience than the detailers and they assist the squad foreman in laying out new work preparatory to ordering the material. The term "draftsman" has a double meaning. Some limit its use to detailers because they do the actual drafting, while others refer to everyone in the Drafting Department as draftsmen, particularly those who have reached or passed the rank of detailer.

1. **Method of Procedure.** — When a new contract is received in the Drafting Department the Chief Draftsman studies the general character of the structure, notes the time limit if any, and assigns the work to the squad foreman who can best handle it. The squad foreman makes a careful study of the whole contract and adopts the method of procedure by which the work under his direction can be carried out most efficiently. In general his first aim is to have all main material ordered as soon as possible because of the usual delay at the mills. In some types of structure either he or his more experienced men can list most of the material directly from the design sheets. In certain classes of work such as truss work it may become necessary to make small layouts of connections, or even to begin the working drawings in order to determine the lengths of the material; later these preliminary drawings may be given to detailers for completion. In other types of structure such as office buildings it may prove feasible to make the plans and diagrams before listing the material. These diagrams may be made so complete that the material may be listed from them quickly and accurately, and the detailed drawings may become routine work which can be done by men of limited experience. The preliminary lists of material are usually sent to an Order Department where the material is summarized and the short pieces are grouped in multiple lengths to be cut after they arrive from the mill. The squad foreman divides the drafting among the detailers to the best advantage so that the work may be carried on efficiently and in logical order. Part of the drawings may be sent to the shop before all are completed and as far as practical an attempt should be made to complete the drawings in approximately the same order that the corresponding members will be erected. For example, the drawings of the ground floor beams in an office building should be made before the drawings of the roof beams. Erection diagrams should be made as soon as possible so that the marks of all members may be recorded as soon as determined. Drawings should be checked shortly after they are made and the shop bill for each drawing should be made as soon as the drawing is completely checked. A shop bill is a summary of all the material required to make all the members represented by a drawing. Later shipping bills and lists of rivets and blots to be used in erection are prepared.

2. The squad foreman should keep **progress sheets** so that he and the chief draftsman can tell what has been done, what is being done, and when and where the blueprints have been issued. He should keep separate files for the drawings and for the correspondence of each different contract. Usually all communications pass through the hands of the chief draftsman who writes all letters and serves as intermediary between the men in the drafting room and those outside. A detailed account of the duties of the draftsmen appears in Parts II and III.

3. There should be greater **coöperation** between the engineering department and other departments than sometimes exists. Shop men and erectors often complain of points in the design or the details of a member which bother them, but seldom do their criticisms reach the source of the trouble. It would be to the advantage of all concerned if the erectors and the shop foremen would issue periodical bulletins which would reach each designer, each draftsman, and each shop foreman. Suggestions and constructive criticisms could thus be presented in such a manner that future trouble might be avoided.

CHAPTER III

THE MANUFACTURE OF STRUCTURAL STEEL *

Synopsis: This topic has no direct bearing upon the work of the draftsman but he should have a general idea of how steel is made. It is important that he know the form it is in when it first reaches the structural company.

1. **Iron.** — Most of the iron ore is taken from open pit mines in the Lake Superior region and shipped by boat and by rail to blast furnaces where it is smelted or reduced. The ores are oxides of iron and the reducing agent is carbon. A blast furnace is in continuous operation. It is charged at the top and the molten iron flows by gravity to the bottom where it accumulates until tapped at intervals of about 6 hours. Besides the ore the charge includes a flux, and the reducing agent in the form of coke which serves also as fuel. An intense heat is maintained by means of a continuous hot air blast. The blast is heated by being passed through a "stove" lined with fire brick. Four stoves are used in turn, the ones not in use being heated by the hot gases from the furnace. Limestone is the flux commonly used to unite with the impurities freed from the ore to form a fused mass called "slag." This floats on top of the iron and is tapped at a higher elevation. The iron tapped from a blast furnace is called "pig iron" because it is often cast into "pigs" of about 100 pounds for convenient handling. Pig iron is used in iron foundries for making iron castings, but most of the pig iron is made into wrought iron or steel. When the steel furnaces are near the blast furnace the iron may be transferred in the molten state in large ladles. Pig iron contains small quantities of carbon (3 to 4 per cent), silicon, sulphur, phosphorous, and manganese. It has so much carbon that it is not malleable at any temperature. The capacity of a blast furnace is about 500 tons of iron per 24 hours.

2. **Structural steel** is made by the "open hearth" process. A higher grade of steel for tools and instruments is made in crucibles or in electric furnaces. At first structural steel was made by the Bessemer process in a converter of from 10 to 20 tons capacity. A cold air blast was used for about 10 minutes after which the steel was ready to be poured. Bessemer steel is inferior in quality and is less reliable than open hearth steel, and the Bessemer process has been largely superceded by the open hearth process. In the latter the charge is placed on a shallow hearth and subjected to a hot air blast. The charge includes besides the pig iron, iron ore, steel scrap, and usually limestone. Gas is used for fuel, and when the charge is melted the flux rises to the top. This flux contains the iron ore which forms a blanket to prevent the oxygen of the air from combining with the iron. The impurities in the iron become oxidized by the iron ore which in turn receives new oxygen from the air. The percentage of carbon and phosphorus is thus reduced. The steel is tapped into large ladles where it is "recarbonized" by adding the proper amount of carbon and other ingredients to give the desired quality. Structural steel contains from 0.15 to 0.3 per cent of carbon. The capacity of an open hearth furnace is from 30 to 70 tons, and the operation requires from 6 to 10 hours.

3. **Rolling the Steel.** — From the ladles the steel is poured into ingot molds and allowed to cool until sufficient crust is formed to permit handling. The molds are then stripped off and the ingots are placed in

* The manufacture of iron and steel is treated more completely in many books, as for example: Stoughton's "The Metallurgy of Iron and Steel," McGraw-Hill Book Co. Inc., New York; Moore's "Materials of Engineering," McGraw-Hill Book Co. Inc., New York; or Burt's "Steel Construction," American Technical Society, Chicago.

ovens called "soaking pits" until the inside portion solidifies and the whole becomes of the proper uniform temperature for rolling. Structural shapes are formed by passing the material between rolls of the proper shape, the cross section being reduced and the piece elongated at each pass. The ingot is first passed between the two cylindrical rolls of a "blooming mill" and flattened. The rolls are then reversed and placed closer together and the material is passed between them in the opposite direction. In this manner slabs are made of suitable size to be placed in another mill to be rolled into plates. For other shapes, the ingot is rotated at right angles between successive passes to form a "bloom" of the size best adapted to the shape of the "roughing rolls." The blooms are cut into lengths which will result in the proper lengths of the final sections. The roughing rolls are grooved to work the blooms down gradually to shapes which approximate the finished pieces, then "finishing rolls" are used. Both the roughing and finishing rolls are so shaped that at each pass the material is reduced in cross section to approach the finished shape. These rolls are "three high" so that they need not be reversed, the material passing alternately between the lower two in one direction and the upper two in the opposite direction. Typical roughing rolls for an I-beam are illustrated in Fig. 24 (*a*), and finishing rolls for an angle in Fig. 24 (*b*). The material is handled on roller platforms on both sides of the rolls. These platforms may be raised or lowered to receive the steel at one elevation and deliver it at another. The material may be moved longitudinally by means of the rollers, and transversely by means of arms between the rolls. The whole operation is controlled electrically.

Fig. 24 (*a*). Typical Roughing Rolls for an I-beam.
(*Courtesy of the Pittsburgh Rolls Corporation.*)

Fig. 24 (*b*). Typical Finishing Rolls for an Angle.
(*Courtesy of the Pittsburgh Rolls Corporation.*)

1. **The Effect of Spreading the Rolls.** — The finishing rolls are kept at a fixed distance apart during the rolling. When they are spaced at the minimum distance the lightest section of a group is made. When the rolls are spaced farther apart heavier sections are made, as indicated in Fig. 25 (*a*). The effect of spreading the rolls is to increase the web thickness and the flange width of an I-beam or channel without changing the depth. The variations are indicated in the tables. Angles of different thicknesses may be rolled in the same mill but from Fig. 25 (*a*) it is apparent that as the rolls are separated the lengths of the legs as well as the thickness will be increased. The tables of this book conform to the usual practice and indicate only the nominal lengths of legs. Since the nominal size is the size only of the thinnest section of a group, the draftsman should take care to allow ample clearances for the overrun of the thicker angles. The increase in the length of the leg is equal to the increase in thickness. On account of this variation and also because of the wearing of the rolls and of the inaccuracies in setting them, the lengths of angle legs and the widths of I-beam and channel flanges are indefinite; dimensions should be referred to the backs of angles and channels and to the center lines of I-beams rather than to the rolled edges.

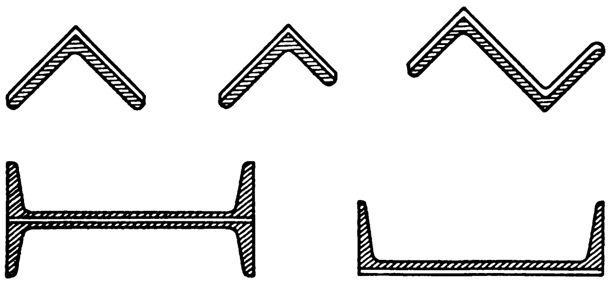

Fig. 25 (*a*). The Method of Enlarging Cross Sections by Spreading the Rolls.

2. **Mill Variation.** — Structural shapes other than plates are sawed to the ordered lengths as soon as they leave the finishing rolls, while still red hot. They must be measured and sawed into the proper number of pieces before the following piece leaves the rolls, and extreme accuracy cannot be assured. The usual "mill variation" is plus or minus $\frac{3}{8}$ of an inch, and drawings should be so made that I-beams and channels particularly can be used without being recut in the shop. In view of this variation, the mill orders should be given in lengths which are multiples of $\frac{1}{4}$ inch and preferably $\frac{1}{2}$ inch. The maximum length varies with the size from 50 to 90 feet.*

3. **Plates** may be rolled in a mill with horizontal rolls only, or in a Universal Mill which has vertical as well as horizontal rolls. The horizontal rolls are brought closer together between successive passes of the plate until the desired thickness is reached. At each pass the length is increased, and unless vertical rolls are used the width also is increased. "Universal Mill" plates or "edged plates" having rolled edges are rolled up to 48 inches in width and in some sizes up to 100 feet in length.* All other plates have to be sheared to the desired width and they are termed "sheared plates." These are rolled up to 132 inches in width and the lengths vary inversely with the width from 15 to 47 feet.* Universal Mill (UM) plates are used chiefly for the cover plates of bridge girders; sheared plates are used for most other work. Sheared plates are usually furnished unless Universal Mill plates are specified. Plate thicknesses vary by sixteenths of an inch from $\frac{3}{16}$ to 2 inches, the usual

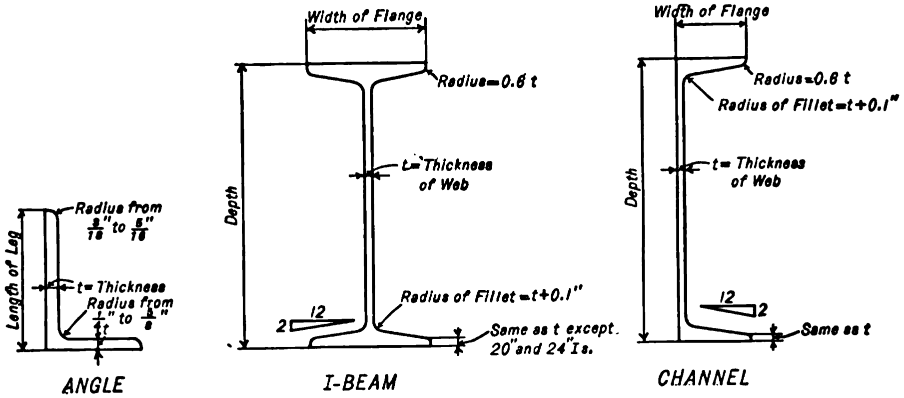

Fig. 25 (*b*). Actual shapes of cross sections.

range in structural work being from $\frac{5}{16}$ to 1 inch except for fillers and bearing plates. Any width of plate may be sheared from larger sizes,

* See tables of extreme lengths in Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co. Inc., New York.

but the draftsman should attempt to use widths without fractions, and if over 24 inches in multiples of 2 inches. The reason for this is to reduce the number of different sizes, which expedites the filling of an order at the mill and simplifies the maintenance of a serviceable plant stock.

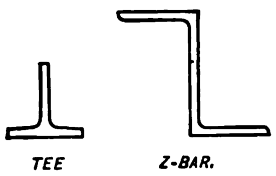

Fig. 26.

1. **The actual shapes** of the cross sections of the angle, the standard I-beam, and the channel are shown in Fig. 25 (*b*). Plates, and round and square rods need not be illustrated. Shapes used less commonly than formerly are shown in Fig. 26. The proportions of rails are shown on page 317. Curves are introduced to facilitate rolling. These curves are disregarded in computing the areas of cross section and the weights per foot of length. The curves are not shown in the usual structural drawings, but the draftsman must constantly bear in mind their existence and make proper allowance. This is especially important in the places where the introduction of curves results in the use of more metal rather than less, as for example at the junction of the flanges and the web. Such additional metal is termed a "fillet" and it serves also to reinforce the web at critical points.

CHAPTER IV

THE FABRICATION OF STRUCTURAL STEEL

SYNOPSIS: This outline of the shop operations of a structural company is given primarily for students. A more thorough knowledge gained from observation is essential to draftsmen.

1. In order to make working drawings intelligently, it is necessary that a person have a very definite conception of their purpose and of the use to be made of them. To those who have had no opportunity to gain this knowledge this chapter gives an abstract of the more important steps taken in preparing or "fabricating" the steel for erection. The term "**fabrication**" is interpreted to include all the shop work necessary to lay out, cut, punch, assemble, and rivet into complete members, the steel shapes as they come from the rolling mills. In general the steel shapes are handled cold during fabrication in contrast to their being heated at the steel mills. A member may be a single piece as a beam or an angle, or it may be composed of many pieces as a girder or a truss. Shop work is preferred to field work because of better facilities; the work at the site should be reduced as much as practical considering the limits in the size of members due to the requirements of shipment and erection.

2. Only the more **elementary points** can be mentioned here,* but students or others who seek employment as draftsmen must make a careful study of shop methods. It is sufficient for the more observing men to make frequent trips to the shop outside of their office hours, cultivating the habit of learning something new at each trip even in short visits during the noon hour. Many men wander blindly through the shop day after day without tangible results. For them a limited period of shop work or field work in the capacity of timekeeper may prove beneficial.

3. **Shop methods** are to a certain extent dependent upon the size of the company, its equipment, and the nature of its output. In general most of the drawings are first used in the templet shop and then in the structural shop. In some cases, such as simple beam work, the measurements may be laid out in the structural shop directly. Drawings for special work, such as eye bars, castings, rollers and pins, and forgings, are sent directly to the eye-bar shop, to the pattern shop and foundry, to the machine shop, or to the forge shop. Some companies are not equipped to handle all of this work but have it done elsewhere when occasion arises.

4. **The plant layout** should be such that the material may pass from the receiving or stock yard to the shipping yard through the different operations with the least movement. The different machines should be arranged so that the material may pass from one to the other in logical order without interference. The material is handled by overhead traveling cranes, jib cranes, gantry cranes, small hoists, derricks, or small trucks or cars on narrow-gage or standard-gage tracks.

5. **In the templet shop** "templets" are made for the component parts of almost all plate and angle work and for some I-beams and channels. The templets are wooden strips or skeleton frames made of strips, upon which are shown the location of all holes and cuts. They are usually made from $\frac{7}{8}''$ or $\frac{3}{4}''$ planed pine boards. Templets for angles are made of one or two strips. (See Fig. 28 (*a*) and (*b*).) The single strip may or may not be the same width as the corresponding angle leg, but transverse

* See also Thayer's "Structural Design," Vol. I, D. Van Nostrand Co., New York; Merriman and Jacoby's "Roofs and Bridges," Vol. III, John Wiley and Sons, Inc., New York; or Well's "Steel Bridge Designing," The McGraw-Hill Book Co. Inc., New York.

distances must be measured from the edge of the strip which will be placed at the vertex of the angle. The positions of all holes for shop or field rivets are carefully laid out on the templet and holes ½″ in diameter are bored in the wood through which the centers are indicated on the steel by means of a "center punch." When holes are to be punched in both legs of an angle one strip may be used for both or the templet may be made of two strips fastened together at right angles to fit over the steel angle. Distances are measured from the inner vertex to correspond to distances on the angle which are measured from the outer vertex. One strip must be full length but the other may be made of one or more short pieces instead of full length if the number of holes is small. When angles are made in pairs, i.e., rights and lefts (page 81 : 2) as stiffening angles or flange angles, a T-shaped templet is made. See Fig. 28 (c). One-half is used for one angle and the other half for the other angle, the holes in the stem serving for one leg of each angle. For small plates full sized templets are used, but for large plates rigid frames are made from strips of wood in order to save lumber and to facilitate handling. Parallel strips may be used for the holes along the edges of a plate, cross strips being inserted wherever intermediate holes are required. Sufficient diagonals are added to prevent distortion. For small connection or "gusset" plates templets are often made of cardboard. This is more easily cut and punched than wood and it is sufficiently durable to be used many times. For I-beams and channels several types of templets may be used. For the flanges strip templets may be used if the number of similar pieces justifies their use; otherwise the holes are laid out directly on the steel. The holes in the web may be (1) laid out on the steel, (2) laid out on full length templets, or (3) laid out in groups on separate templets which are located by direct measurement. The third scheme is well adapted to the use of standard connections, for the group templets may be used repeatedly for different contracts. In order to economize lumber and to reduce the number of templets to be handled and stored, one templet may often be made for pieces which are not identical. The pieces must be similar even though of different length, and most of the holes must be in the same relative position although not all of the holes need be punched in every piece. Special marks and notes are painted on the templets to indicate which cuts and which holes are to be used for each piece. The contract number, the drawing number, the size of the holes, the identification mark, and the number of pieces required, are painted on each templet. Since the detailed dimensions of a working drawing are used chiefly by templet makers, the draftsmen should pay particular attention to their needs.

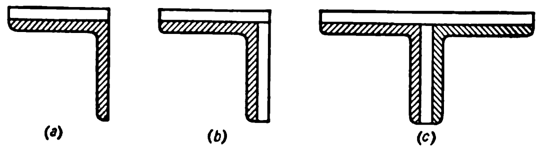

Fig. 28. Templets for Angles.

1. **The Stock Yard.** — As the steel arrives from the rolling mill it is unloaded in the receiving yard or stock yard by overhead cranes. Usually enough material of different shapes is kept in the stock yard for the details and other parts which cannot be included in the original mill orders. Material required on account of changes or additions to a structure may often be provided in this way without delay. Most of the material for each contract is ordered specially for that contract and it is held in the receiving yard until required. Since much of the material is ordered in multiple lengths, particularly plates and angles, it must be cut to dimension. This is usually done in the receiving yard or in the receiving end of the structural shop from the "shop bills" upon which the required material is summarized. Plates and angles are cut cold by **shearing,** each by a single stroke. The plate shear has one fixed horizontal blade and one movable blade which moves in a vertical plane. One end of the movable blade is slightly lower than the other so as to cut the plate gradually instead of the whole width at once. The cutting edge of the upper blade is placed so it will just clear the cutting edge of the lower blade much as the two blades of ordinary scissors are arranged. Angle shears cut both legs at once, one leg being placed against a horizontal cutting edge and the other against a vertical one. A single knife with two cutting edges moves diagonally past these two. Diagonal cuts on plates and angles may be made by running in the material obliquely or by rotating the shears horizontally. I-beams and channels are usually ordered from the mill in the desired lengths because the flanges prevent them from being cut as simply as plates and angles. Special beam shears are made which cut first one flange and then the other but nearly an inch

of material is wasted at each cut. Circular "cold saws" are also used for sawing beams, especially the heavier ones and those with diagonal cuts. The oxy-acetylene flame is sometimes used when shears cannot be used advantageously.

1. Some of the material becomes bent during shipment, particularly the plates. Before they can be used in the shop they must be straightened by being passed between a series of **straightening rolls.**

2. The first step in the fabrication of riveted work after the material is cut to length is the "**laying out.**" This includes the marking of the steel from the templets when templets are used, and also the laying out on the steel directly from the drawings when templets are not used. The wooden templets are clamped in the proper position on the steel and all notches and special cuts are marked with a piece of soapstone. The centers of holes are indicated by small dents in the steel. These dents are made by placing a "center punch" through the $\frac{1}{2}''$ holes of the templets and striking the top of it with a hammer. The center punch is slightly smaller than the hole in the wood so that it can be centered easily, and a short sharp point protrudes from the end for making the dent. The soapstone marks which indicate the cuts are made more permanent by a series of center punch marks. White paint is used to call attention to the presence of all marks which might otherwise be overlooked. The contract number, the identification mark, the sheet number of the corresponding drawing, and the size of the holes are also painted on the steel, the first two being painted on at the mill or in the yard where the material is cut to length.

3. **Coping.** — Corners of plates and angles may be cut off by the shears already described. Notches may be cut with special "blocking out" punches or by punching a series of circular holes and then smoothing the rough edges with a pneumatic chisel. Reëntrant angles in plates should be avoided for they are difficult and expensive to make. I-beams and channels are coped and notched in a special coping machine or by means of an oxy-acetylene flame. A beam is "coped" at the end to clear the flange of a beam to which it connects. First the flange is blocked out and then the web is cut. Coping requires two or three strokes; some machines block out only one side of the flange of an I-beam at a stroke while others cut both sides. The flanges may be blocked out on one side only if desired.

4. **Other preliminary operations** may be required in shaping the component parts of a member before they can be assembled. For example, the ends of "stiffening angles" which are to be placed against the inner faces of other angles must be rounded to allow for the curved fillets of the second angles (page 96 : 1). This may be effected (1) by grinding, (2) by planing, or (3) by chamfering with rotating cutters. When a stiffening angle is "crimped" or offset for one leg of the angle against which it is to rest, as shown in section BB, Fig. 102, it must be taken to the forge shop and bent while hot. Plates or other shapes occasionally have to be planed on the edges, and some plates must be planed on one side to insure a uniform bearing surface. A small cutting tool is made to pass across the surface repeatedly thus cutting a series of thin narrow strips until the desired depth or area is planed.

5. **Punching.** — Holes may be punched, drilled, or bored. Rivet holes are usually punched to the desired size by "punches." The shape of the punch and the corresponding die is shown in Fig. 29. The punch housing is stationary; the steel to be punched is moved with the aid of a small hoist until the dent which indicates the location of a hole is directly below the small projection on the punch shown in the figure; the punch is then released and it passes through the metal. Most holes "are single punched," i.e., punched one at a time, but the larger plants are equipped with some form of multiple punch, in which any or all of the punches can be released at once. For example, multiple punches are used for punching groups of holes for standard connections in beams, and also for punching the holes in web plates and cover plates. In conjunction with the latter a spacing rack is sometimes used which can be set and used in advancing the plate to correspond to the desired rivet spacing. Special racks are sometimes used in conjunction with an ordinary punch for duplicating connection plates. The holes are first laid out on one plate and punched, then this plate is clamped in the rack under an indicator. A blank plate of the same form is clamped in the rack in such a position that the punch is in the same relative position as the indicator of the first

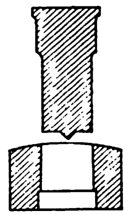

Fig. 29.

plate. The rack is moved until the indicator falls into a hole, then the punch is released and a corresponding hole is made in the second plate. Obviously no benefit is derived from a multiple punch unless it can be used many times with each setting.

1. **Drilling.** — Rivet holes are also drilled. In some of the highest class of work drilled holes are specified for they may be made true cylinders, accurately centered, with less damage to the surrounding metal than punched holes. Holes for important field connections are often drilled in both connecting parts through the same metal templet so that they will match exactly. Sometimes connecting members are held together in the proper relative position and the holes are then drilled through both members at once to insure a perfect connection. Holes are also drilled in metal which is too thick to be punched. In general holes may be punched in metal as thick as the diameter of the punch, but it is difficult to prevent the punches from bending when the metal is thicker. Many specifications state that no holes shall be punched in metal thicker than $\frac{3}{4}''$. Fixed drill presses are used for small pieces such as base plates which are moved into position under the drills. Radial drills may be moved radially in an arm which can be swung about a vertical axis. Gang drills are used in some plants for drilling a large number of holes in heavy members; these are groups of radial drills mounted in a gantry frame which passes over the material to be drilled.

2. **Sub-punching.** — Since holes in connecting parts are not always in perfect alignment, and since drilled holes are much more expensive, the holes in first class work are often sub-punched and reamed. The holes in each component part are punched about $\frac{1}{4}''$ smaller than the desired diameter, then after the parts are assembled pneumatic reamers are used to ream out the holes to the proper size.

3. **Assembling.** — After the component parts of a member are ready they are assembled by "fitters" and held in position by shop bolts. These bolts are usually longer than necessary, enough washers being used under the nuts to save time in tightening them. At least two bolts are used in each piece to keep it from twisting.

4. **Riveting.** — The assembled members are next taken to the riveters who drive the red hot "shop rivets." These rivets may be driven (1) by fixed hydraulic or compressed air riveters, (2) by movable compressed air riveters or "riveting bulls" such as shown in Fig. 30 (*a*), (3) by pneumatic hammers shown in Fig. 30 (*b*), or (4) by sledges. Each rivet is made with one head; the other head is formed after the rivet is put into position. The diameter of the hole is made about $\frac{1}{16}''$ larger than the diameter of the shank of the rivet, so that the rivet can be inserted more easily; when a rivet is properly "driven" the new head should be well formed and centered and the shank should be "upset" to fill the hole. The rivets driven by machine are worth more than those driven by hammer because the enormous pressure (sometimes 100 tons) is sufficient not only to form good heads and to upset the shanks completely, but to bring and hold the component parts in perfect contact while the rivets are being set. Some "fixed" riveters can be raised or lowered to drive rivets in girders and similar members at different elevations as the members are moved along. Perhaps the majority of rivets are driven by movable riveters, either the member or the riveter being moved as is most convenient. Air hammers or sledges are used for driving rivets which are inacessible to the other

Fig. 30 (*a*). Riveting Machine.
(*Courtesy of the Vulcan Engineering Sales Company.*)

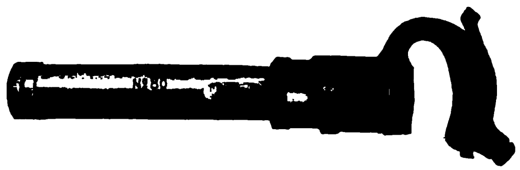

Fig. 30 (*b*). Pneumatic Hammer.
(*Courtesy of the Chicago Pneumatic Tool Company.*)

forms of riveters. When hammers are used, the rivets are held in place by "dolly bars" either of pneumatic type or simply bent rods with cup-shaped ends to fit the rivet heads. Countersunk and flattened rivets are driven by the same methods by substituting flat dies for those which form the "button heads" of the ordinary rivets. Holes for countersunk rivets are reamed by special reaming tools placed in the drill presses or in the pneumatic reamers. Projecting portions of countersunk rivet heads may be chipped off, if necessary, by means of a cutting tool or chisel placed in the pneumatic hammer.

1. **Milling.** — The ends of members which are expected to transmit stress by direct bearing must be "planed," "faced," or "milled." This is done by means of a "milling machine" or "rotary planer" which has numerous cutters arranged near the edge of a circular cutting head. As the head slowly rotates it advances across the end of the member. Unlike the teeth of a saw, the cutters are set in the plane face of the head toward the member to be cut instead of in the periphery, and it is important that they project equal distances from the face. The member is clamped in front of the planer and the latter can be turned horizontally to cut the member at the proper angle. Milling machines are often used in pairs so that both ends of a member may be faced at once.

2. **Boring.** — Holes for the pins of pin-connected structures are bored by large boring machines. These are often arranged in pairs so that the holes at each end may be bored simultaneously at the proper distance apart. The holes are usually roughed out by punching either a large hole or a series of small ones, so that the shaft which holds the cutter may pass through. The cutter of the boring machine then enlarges the hole to the proper size.

3. **Inspection.** — Each member is inspected when the shop work is completed. The inspector checks the important measurements and the field connections. He makes sure that every connection is provided for, and in general that the member is complete and properly made. He tests the rivets with a hammer and rejects any loose ones.

4. **Painting.** — One or more coats of paint or oil are applied to each member before it is shipped. Surfaces which are inaccessible for painting after the member is completed are given one coat of paint before the parts are assembled.

5. **Shipping.** — Most members are first shipped by rail on open cars. They are loaded to the best advantage, the larger members being separated by wooden blocks to prevent damage during shipment. Long girders and chords may extend over two or three flat cars but they must be so mounted that they will not interfere with the passage of the train around curves. Many of the smaller loose pieces are bolted to larger members for shipment in accordance with notes on the drawings; others are wired together or boxed.

6. **Other operations** are required to supplement those just described. Some plants are fully equipped while others have much of the miscellaneous work done elsewhere. Among the most important departments may be mentioned the machine shop, the forge shop, the foundry, the pattern shop, the eye-bar shop, and the bolt, nut, and rivet shop. The machine shop provides for repairs and shop maintenance as well as for the special machine work required in the structures made for the customers. Here the various tools are sharpened and kept in good condition, and often some tools are made. Pins, rollers, and turned bolts are made in the machine shop, bed plates are planed, and castings are drilled and finished there. In the forge shop angles and other shapes are bent, rods are upset at the ends to neutralize the effect of thread cutting so that the full cross section of the rods will be developed, and loop rods, clevises, turnbuckles, etc., are made. In the foundry all steel and iron castings are made, as for example, column bases, bridge pedestals, and beveled washers. Patterns are used in making sand molds into which the molten iron or steel is poured. Patterns are made in a pattern shop by pattern makers not by templet makers for they require an entirely different class of workmanship. Patterns are models of the finished castings, made carefully to scale. They are made larger than the castings by the use of "shrinkage scales" to allow for the shrinking of the metal while cooling. In the eye-bar shop the ends of eye bars are upset in large hydraulic presses which form the heads. The pin holes are then punched and later bored to the proper size. The bars are then annealed, i.e., heated and allowed to cool slowly in order to restore the uniform properties of the steel before the ends were heated and upset. Rivets and bolts are made from heated rods in special rivet and bolt upsetting machines. Nuts are punched from flat bars. After the bolts and nuts are cool they are threaded. The

separate departments mentioned in this paragraph are referred to as "shops" but they do not necessarily require separate buildings. For example, the pattern shop and the templet shop may be under one roof and the eye bars and the rivets and bolts may be made in the forge shop. It will be noted that similar operations are carried on in different departments, but this is a matter of plant economy. For instance, it is cheaper to maintain milling machines and boring machines in the main structural shop or in the eye-bar shop as well as in the machine shop than it is to carry all of the heavy members to the machine shop for milling.

PART II — STRUCTURAL DRAFTING

CHAPTER V

STRUCTURAL DRAWINGS — THE DRAWING

SYNOPSIS: In the first chapters of this Part II are given the fundamental principles of drawing. In this chapter are discussed the projection, the arrangement and the selection of views, the working units, and the scale.

1. **A Structural Drawing** is a working drawing for steel construction, and it should fully represent one or more members of a bridge, a building, or other similar structure. It is made primarily for use in the templet shop and the structural shop, and it should not only give all the information necessary for the construction of the members in the shop, but also provide for their proper connection to other members in the field. Each member is composed either of a single piece of structural steel or a combination of the common rolled steel shapes, and for this reason many methods are necessarily used which are peculiar to this form of drawing — methods that are somewhat different from those used in other kinds of working drawings. It is essential that structural drawings be made and checked carefully, for the cost of replacing ruined pieces is great, and the resulting delay is expensive.

2. A structural drawing may be subdivided into the following **elements,** which will be discussed in separate chapters:

The Drawing
The Conventional Methods of Representation
The Conventional Methods of Billing
The Dimensions
The Notes, the Title, and the Border.

THE DRAWING

3. **Projection.** — Working drawings are drawn in orthographic projection, and the third angle of projection is used. At times a knowledge of isometric, oblique, and cabinet projections, and of perspective drawing is of benefit to the draftsman, although usually not essential. In orthographic projection, a "view" is not a true view as in perspective drawing, but it represents the projection, by parallel lines, of one face and the projecting parts of a member upon either a horizontal or a vertical plane. A view which is projected upon a horizontal plane is termed a **plan,** while a view projected upon a vertical plane is termed an **elevation.** These terms are commonly used in erection diagrams, "show drawings," and design sheets, where the structure is treated as a whole, but they are not often applied to working drawings in which individual members are considered.

4. **The proper views** of any member should be selected to represent that member to the best advantage. Two or more views are usually necessary, but no more views should be drawn than those required to show clearly the arrangement of the different pieces of which the member is composed, together with all the necessary dimensions. Much time and space may be wasted by drawing unnecessary views. The usual

views employed are: — the front view or front elevation, the top view, the bottom section, and the end views. A rear view becomes necessary in some cases, and frequently one or more sectional views.

1. All views must bear a proper relation to one another as in Mechanical Drawing; see Fig. 34. The **top view** must be placed directly above the **front view,** and an **end view** opposite the front view near the end it represents. It is customary in making structural drawings to replace the bottom view by a **bottom sectional view** placed directly below the front view, but drawn *as seen from above.* The section plane is assumed to be passed just above the bottom details which are to be shown, in such a manner that comparatively few parts are cut. See page 37 : 2.

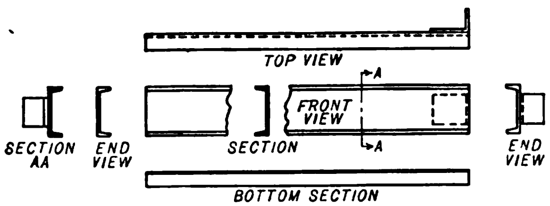

Fig. 34. Arrangement of Views.

The reason for substituting a bottom section for a bottom view is to obtain a better correlation between it and the top view, so that it will be more apparent whether a connection on one side of the top flange and a connection on the bottom flange are on the same or the opposite sides of the member.

A sectional view, other than the bottom section, may be placed anywhere on the sheet, provided the position of the section is clearly indicated. It is the usual practice, however, to place the sectional view either at the end of the member, adjacent to or in place of an end view, or else in a break in the member, preferably in the main or front view at the point where the section is taken. In any case the sectional view must be drawn as seen looking *toward* the parts which are to be shown.

2. **The distances between the different views** of a member are not necessarily equal, but they are made just great enough to provide ample room for dimension lines, explanatory notes, and other lettering which must be put on the sheet. Provision should be made also for any projecting parts, such as gusset plates, base plates, or connection angles.

3. **The Position on the Sheet.** — Most members are shown on the drawing separately, but in some classes of work, as for example truss work, it is convenient to draw several members together, even though they are to be shipped separately, in order to show their relation to each other and to save the duplication of details. (See Fig. 122.) As far as practicable it is well to draw horizontal members horizontally on the sheet, i.e., lengthwise, and to draw vertical members vertically, i.e., from the lower to the upper edges of the sheet. Long columns, or other vertical members which would appear too crowded in this position may be drawn lengthwise of the sheet. Inclined members may be drawn horizontally in order to economize space, unless it is desired to show their relation to other members. It is desirable to draw each member in such a position that the plans and the diagrams can be used to the best advantage, without turning them needlessly.

4. **Parts Shown.** — Only the pieces which are to be shipped with a member should be shown on the detail drawing of that member, although in unusual or complicated work part of a connecting member may be shown in outline, by the use of dotted lines or lines of red ink (page 36 : 2). This should be done for the benefit of other draftsmen or of the erector, only to make more clear a connection which might otherwise be obscure or difficult to understand from the drawing — never if confusion is likely to result in the shop. In drawing any view of a member it is unnecessary to show all the parts, such as connection angles or plates or rivet heads, which project from faces other than the principal one shown, merely for the sake of completeness; as a rule the drawing is thereby complicated, and valuable time is wasted. They should be shown, however, if the drawing is made more clear, or if an extra view may be dispensed with by so doing.

5. If a member is practically **symmetrical** about its center line, it is necessary to show only one-half of the member with a note to the effect that it is "Symmetrical about the center line," or "Symmetrical about the center line except as shown or noted," or that "The other half is exactly like the half shown." It is customary to show the left end of the member. See page 86 : 5. By this method much excessive duplication may be avoided. Simple members are usually shown completely.

6. **The usual working units** in structural work are feet and inches and fractions of inches expressed to the nearest sixteenth. Since many of the tapes used in the shop are subdivided to eighths only, it is more con-

venient for all concerned to use multiples of ⅛″, or preferably ¼″, whenever practicable, it being understood that each dimension is correct to within $\frac{1}{32}$″, i.e., expressed to the nearest 16th. In a few instances it may become necessary to employ dimensions in 32nds, where dimensions are subdivided, or in 64ths, or in 100ths, for the sizes of pin holes.

1. In order to obtain the best arrangement and appearance of a drawing, **a preliminary freehand sketch** of each member should be drawn on a separate sheet. This sketch should show the number and the arrangement of the views, the main dimensions, the number of dimension lines, and the position of the principal connections. The sketch need not be elaborate, but it should enable the draftsman to choose the best scale, to determine the distance between the different views, to plan the types of connections to use, and to report any missing information so that it may be supplied with the least possible delay. It will usually be found that the preparation of this sketch will actually save more time than is consumed in its preparation, and in addition the work will be arranged more advantageously.

2. Although structural drawings are for the most part drawn approximately to scale, it is unnecessary and undesirable to employ the same degree of accuracy used in map work; one is never permitted to scale a drawing to obtain a dimension, but is required to use the figured dimensions only. Time should not be wasted, therefore, in **too accurate plotting,** but greater stress should be laid upon giving the dimensions correctly. The more complicated drawings are drawn to a definite scale to avoid crowding, and to simplify the addition of other connections. In simple drawings, however, only the details are scaled, and the lengths between details are shortened to save space. Beams are usually drawn upon printed forms which determine the outline regardless of the actual dimensions required (page 83 : 4). The details may be drawn to approximately the same scale as the depth of the beam, and the distances between details estimated roughly proportional to the length. In columns or other long members it is sometimes convenient to use one scale for the details and a smaller scale for the distances between details, although these distances may frequently be estimated. Small distances, such as the thicknesses of angles or plates, are often exaggerated on small scale drawings to prevent the lines from running together. After a little experience the draftsman will be able to estimate small unimportant distances without the scale.

3. **The Scale.** — Structural draftsmen use Architects' scales for the most part, although they have more or less use for Engineers' (decimal) scales also. The adoption of the best scale for the drawing of a large member depends upon the size of the piece, the size of the sheet, the number of the views, and the number of the dimension lines required. In some cases it becomes necessary to use more than one sheet to show a member properly. It is seldom practical to use less than ½″ = 1′ except for plans and diagrams. The scale most used is ¾″ = 1′, but on small or complicated drawings the draftsmen often prefer to use 1″ = 1′. The scales 1½″ = 1′ and 3″ = 1′ are used only for enlarged details or layouts.

Either flat or triangular scales of different lengths and with different graduations may be obtained. Scales one foot long are most used. Triangular ones have all the required scales on one piece, but some draftsmen prefer several flat scales instead. A scale guard should be used with a triangular scale to prevent mixing different scales on one drawing.

The scales are graduated in such a manner that dimensions may be plotted directly without conversion. The main numbered divisions of any of the scales represent feet, and the end foot is subdivided into inches and fractions of inches. Note that the zero mark for both the feet and the inches is at the end of this first foot instead of at the end of the scale. Dimensions less than one foot may be plotted to a given scale by means of the graduated end foot much as they would be laid off full size with an ordinary foot-rule. Dimensions over one foot may be plotted at one setting of the scale, provided the scale is long enough, by placing the proper foot-mark at a given point and by plotting the other point opposite the proper fraction of an inch. For example, to plot 4′–5½″ to a scale of ¾″ = 1′ select the proper edge of the scale graduated to ¾″ = 1′ and place the number 4 (4 ft. from the zero mark) at one end of the required distance and plot the other end 5½ one-inch divisions beyond the zero mark (i.e., 6½″ from the end of the scale). Fractions smaller than those represented by the smallest subdivisions may be interpolated.

4. **The Size of the Drawings.** — Most structural companies have adopted sheets of standard size for their drawings, and with few exceptions all drawings are made to conform to these sizes. The size of sheets most used for principal details and for diagrams is 24″ × 36″. Beams, castings, forge work, bills, and miscellaneous lists are made on smaller forms which vary in size with the different companies. Some companies have one or more intermediate sizes. For student use, "Nor-

mal" drawing paper (15″ × 22″) is well adapted to most work, the sheets of separate-leaf note books being used for smaller drawings, and "Double Elephant" (27″ × 40″), or one-half of "Double Elephant" (cut to 20″ × 27″), for larger drawings. For beam work and billing, small "Drafting Forms" * (8″ × 10½″) have been prepared by the author to correspond to similar sheets in use in the drafting rooms of structural companies. The uses of these forms are illustrated in Chapters XVII, XXIV, and XXVII, pages 83, 146, and 167.

1. **Drawings** are made either **on paper** and then traced, or else **on tracing cloth** directly. The beginner should draw on paper first and then trace his work. Many experienced draftsmen make complete pencil drawings and then have them traced by "tracers" whose time is less valuable. The most satisfactory method for an experienced draftsman is to make the drawing directly on the cloth. He is thus enabled to draw many of the lines in ink without preliminary pencil lines, and consequently much time is saved. Other lines may be penciled on the cloth and later inked. See Chapter XII, page 65.

2. **Black waterproof India ink** is used for structural drawings almost invariably. Colored inks are seldom used. Formerly the dimension lines were made with red ink, but nearly all the structural companies have changed to the use of black, because the red lines do not show distinctly on the blueprints. Since the shopmen work from prints entirely, and use only the given dimensions without scaling, it is important that the dimension lines be clearly defined. Red ink is occasionally adopted to indicate parts of other members which are to connect to the one being drawn, in case the detail is complicated or unusual (page 34 : 4). The red ink is easily seen on the tracings by the draftsmen who are chiefly interested in it, and the lines may be faintly discerned on the prints by the erector without causing any confusion to the shopmen. However, red ink can be erased only with difficulty and some red inks are liable to spread after the drawings are filed in cold vaults; the use of red ink should be limited.

3. The best results in drafting may be obtained by adopting some **systematic method of procedure.** This not only saves time, but also minimizes the chance of making mistakes, or of overlooking connections. Each draftsman may adopt his own method, and different classes of drawings may require different steps, but the following order of procedure will form a guide to the student when making a pencil drawing. Similar suggestions may be found for tracing (page 59 : 9), and for drawing directly on cloth (page 66 : 4).

1st. Make a rough sketch of the member on a separate paper, indicating the position of each connection, thus determining the number of views and the number of dimension lines. It is well to determine the main dimensions also before starting the final drawing.

2nd. Outline the principal view first, and draw the corresponding dimension lines.

3rd. Outline the other views and draw the dimension lines. If preferred, the principal view may be completed first.

4th. Write on the main dimensions.

5th. Draw the details in the order of importance, beginning with the connections to the supports, if any. *Put on each dimension as soon as determined,* indicate the rivets and the holes, bill all the material, and add the necessary notes before proceeding to the next connection.

The draftsman should constantly imagine himself in the place of the men in the shop in order to determine what information is needed on a drawing and how this information can be arranged to be of the greatest service. He should improve every opportunity to watch the shopmen at work, and to become familiar with their methods. He should attempt to build up the drawing of a member in much the same order in which the member itself is built up in the shop, and he should never submit a drawing to be checked until he is satisfied that the members can be constructed as intended.

6th. Complete the dimensions *between* the connections, spacing the necessary rivets or lattice bars.

7th. Note the sizes of the rivets and the holes.

8th. Make the title, and add any general notes which may be required.

4. **A draftsman should always check his own work** as he goes along. He should thus be able to detect any blunders in time to correct them before a large amount of dependent work has been wasted. He can often save more time than he consumes in applying his checks, and in addition he is able to submit to the checker drawings which are comparatively free from mistakes — a point very much in his favor. See page 180 : 1.

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CHAPTER VI

STRUCTURAL DRAWINGS — THE CONVENTIONAL METHODS OF REPRESENTATION

SYNOPSIS: In Structural Drawings the objects are not always represented in exact accordance with the principles of orthographic projection; certain lines are omitted for the sake of simplicity, especially those which represent curved surfaces. Such omissions, and the conventional methods of representing the common structural shapes, are explained in this chapter.

1. **The lines of a drawing** are usually made of two widths; see Fig. 37. The dimension lines, the rivet lines, the projection lines, the working lines, and the cross-section lines are *full black lines* made as *fine* as practicable, but not so fine as to appear ragged on the tracing or on the blueprint. The lines which represent the component parts of the member itself are *black lines* made *heavy* enough to show a contrast with the finer lines noted above.

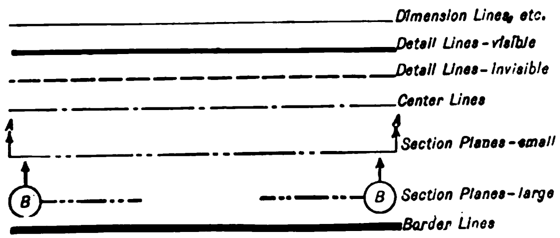

Fig. 37. The Lines of a Drawing.

The lines which represent *visible* edges are made *full*, and those which represent *invisible* edges are made *dashed*. The dashes should be about one-eighth inch long, drawn as close together as practical, but with sufficient space between them so that on blueprints the lines are clearly dashed lines and not full lines. In short lines, or in the lines of small-scale drawings, the dashes may be shorter, and conversely may be lengthened in very long lines, particularly in drawings which are made to a large scale. The appearance of a drawing is greatly improved if large or uneven spaces between the dashes are avoided. Where lines are close together, the dashed lines may be made slightly narrower than the full lines. Dashed lines are liable to be wider than full lines unless the setting of the pen is changed. Center lines are *fine black lines*, similar to dimension lines except that they are made of *dots and dashes* alternating. Lines which indicate where sections are taken, are made of *dashes with two dots between.* These may be made either fine or coarse, depending upon which will show the most contrast with the surrounding lines. They may be continuous, or portions of them may be omitted to avoid confusion with other lines. Arrows should be placed at the ends of these lines to show which part of a cut member is represented in the corresponding sectional view. These lines which indicate a section should be lettered to correspond to the sectional view. Where space permits, circles may be placed at the ends of the lines with the letters inscribed; otherwise the letters may be placed at the ends of the arrows. See Fig. 37. For the use of lines wider than the main lines of the drawing, see pages 37 : 2, and 54 : 7. For the method of inking wide lines see page 59 : 3. *Dotted lines* should be reserved for indicating complicated connections (page 34 : 4).

2. **Sectional views** (page 34 : 1) are usually "cross hatched" or "section lined" so that they may be distinguished from other views. Only the parts which are cut by the section plane (page 34 : 1) should be section lined, and the position of this section plane should be selected

to illustrate the conditions to the best advantage. It is usually best to cut only continuous material so that the details will stand out more clearly. The cross-section lines are fine black lines (see above) drawn at uniform distances apart, inclined 45° with the principal edges of the parts cut. It is preferable to slope the section lines of adjacent component parts in opposite directions in order to accentuate the dividing surfaces. The webs of I-beams and channels which are cut by section planes are usually made solid black for simplicity instead of section lined; similarly web plates are sometimes made solid, as in the bottom sectional views of plate girders.

There are many devices on the market to aid the draftsman in spacing the section lines uniformly, but with a little experience one can produce results which are quite satisfactory for this class of work with simply a T-square and a triangle. If desired, the advance of the triangle can be made more uniform by means of a small strip of wood cut slightly smaller than the opening in the triangle. By alternately holding and sliding the wood and the triangle the latter may be advanced in accordance with the amount of clearance allowed.

1. **Breaks.** — When part of a view is omitted the broken ends should be made so that they may be clearly distinguished from the actual ends. An irregular freehand line may be drawn to represent an imaginary cut across the member, but care should be taken not to draw such a line across a space between component parts where nothing is cut. These irregular lines may be omitted if the limits of each view stand out clearly from the dimension lines and from the other views. When a member is symmetrical about its center line and only one half is detailed, the main lines of the drawing should not be stopped abruptly at the center line but should be extended short distances beyond; in this way one can tell at a glance whether one half or the whole of the member is shown. Typical breaks are shown in Figs. 102, 122, 127, and 143. Occasionally it may be desirable to indicate the true shape of a simple piece at the break in order to dispense with an additional view. This method is illustrated by Fig. 38 (*a*), in which the piece is considered to be cut by a plane at 45°. This method is not recommended for structural drawings although it is sometimes used.

Fig. 38 (*a*).

2. **Curved surfaces** may be indicated by shade lines if it is deemed advisable in order to illucidate the drawing (Fig. 40 (*d*)), but usually this is unnecessary. Some of the surfaces of the common structural shapes are connected by "fillets" with curved surfaces (page 26 : 1), but it is customary to omit these curves from most drawings. Should it become necessary to show the actual forms, the dimensions given in Fig. 25 (*b*) may be used.

3. For the most part a **conventional representation** is shown for each commercial shape of structural steel whether drawn separately or in combination with other shapes. This results in a considerable saving of time and hence of money. The main dimensions are usually scaled, but an experienced draftsman may estimate the spaces between some of the lines, as, for example, those which represent the thickness of metal. As a guide to the student, suggestions are here given to enable him to show each shape in the conventional manner.

4. **The shapes most used** in structural work are plates, angles, I-beams, channels, and round rods. Less frequently, Tees, Z-bars, rails, eye bars, and square rods are used.

5. **A plate** is indicated in any view by a rectangle of the proper dimensions. The width and the length are scaled, but the thickness is usually estimated; the thickness is often exaggerated, if necessary, to show clearly whether a dimension extends to the face of the plate or to the center line.

6. **An angle** is shown in its true shape in the end view (Fig. 38 (*b*)) except that the curves are omitted. (Compare Fig. 25 (*b*).) The front and the top views each have three lines drawn so that they bear the proper relation to the lines of the end view in accordance with the usual principles of orthographic projection. Thus the inner line must be placed near the proper outer line and be made full or dashed according to the position of the outstanding leg. The lengths of the legs are usually scaled, and also the thicknesses of heavy angles; the thickness of an ordinary angle may be estimated.

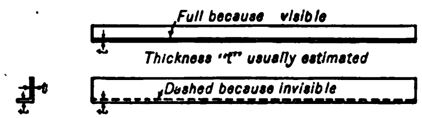

Fig. 38 (*b*). An Angle.

7. **An I-beam** is shown conventionally (Fig. 39 (*a*)) as follows:

End View. — The depth and the flange width are plotted to scale, the flanges being symmetrical about the web. The thickness of the web is estimated and often exaggerated if necessary to indicate clearly whether

a dimension extends to the face of the web or to the center line. The outside edges of each flange are made approximately of the same thickness as the web, and from the points thus located, sloping lines are drawn until they intersect the web lines; all curves are omitted. (Compare Fig. 25 (*b*).) These lines are drawn to a slope of 2 in 12.

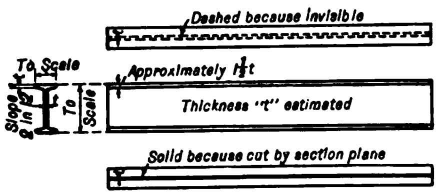

Fig. 39 (*a*). An I-beam.

The lines which represent the inner surfaces of the flanges of all standard I-beams and channels have a slope of 2 in 12. These lines may be drawn most conveniently and uniformly by means of a "beam bevel," which may be bought or easily made of celluloid, wood, or cardboard, in a form similar to that shown in Fig. 39 (*b*). For occasional use a beam bevel may be added to a celluloid triangle either by cutting the bevels on the edge of the central opening, or by scratching one or two sloping lines, as shown in Fig. 39 (*c*). One of these lines may be placed coincident with the flange line (perpendicular to the web) and then the whole triangle may be moved parallel to itself until the edge gives the desired slope line.

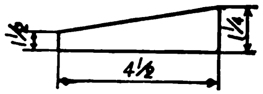

Fig. 39 (*b*).

Front View. — Each flange of the I-beam is represented conventionally by two full lines spaced approximately to show the mean flange thickness. The distance between the outer lines of the two flanges is the depth of the beam to scale; each inner line is drawn so that, if extended, it would cut the sloping lines of the end view about midway between the web and the edges of the flange. The distance between the two lines of each flange is roughly one and one-half times the web thickness.

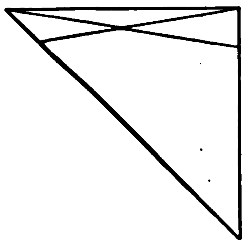

Fig. 39 (*c*).

Top View. — Two full lines are drawn to show the flange width to scale. The web is shown midway between these lines by two dashed (invisible) lines, the web thickness being estimated as before.

Bottom Section. — This is similar to the top view except that the web is shown by a heavy black line equal in width to the web thickness (page 37 : 2).

1. **A channel** is shown conventionally (Fig. 39 (*d*)) as follows:

End View. — The depth and the flange width are plotted to scale. The thickness of the web is estimated and often exaggerated if necessary to indicate clearly whether a dimension extends to the back of the web or to the center line. The edge of each flange is made approximately of the same thickness as the web, and from the point thus located a sloping line is drawn until it intersects the inner web line; all curves are omitted. (Compare Fig. 25 (*b*).) This line is drawn to a slope of 2 in 12. (See note in fine print under I-beam.)

Front View. — Each flange of the channel is represented conventionally by two lines spaced approximately to show the mean flange thickness. The distance between the outer lines of the two flanges is the depth of the channel to scale; each inner line is drawn so that, if extended, it would cut the sloping line of the end view about midway between the web and the edge of the flange. The distance between the two lines of each flange is roughly one and one-half times the web thickness. The inner lines of the flanges are full or dashed, depending upon whether the flanges are on the near side or on the far side of the web; this should correspond to the way the flanges are shown in the end view.

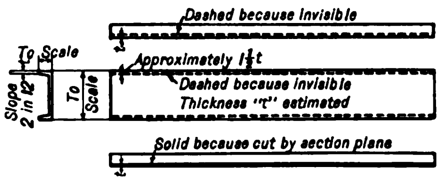

Fig. 39 (*d*). A channel.

Top View. — Two full lines are drawn to show the flange width to scale, one line showing also one face of the web; a single dashed line is added to show the other face of the web, the web thickness being estimated as before. It is important that the web be shown on the proper side of the flange to correspond to the other views.

Bottom Section. — This is similar to the top view except that the web is shown by a heavy line equal in width to the web thickness (page 37 : 2). The web should appear on the same side as in the top view.

2. **Round and square rods** are represented by true views, the ends being shown as circles or squares, and the other views as rectangles. No shade lines are necessary.

1. **A Tee** or T-iron is shown much like an angle, except that the stem is in the middle of the flange, and hence an additional line is required (Fig. 40 (*a*)). Note that both the stem and the flange taper slightly, but in most drawings they may be represented by parallel lines.

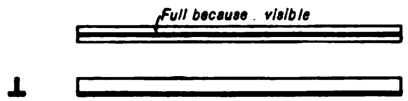

Fig. 40 (*a*). A Tee.

2. **A Z-bar** is shown conventionally (Fig. 40 (*b*)) as follows:

End View. — The depth and the flange width are scaled, but the thickness is estimated and often exaggerated; the thickness of the flanges is uniform and is equal to the web thickness.

Front View. — The depth is shown to scale between two full lines. Additional lines are drawn just inside of these lines to represent the inner edges of the flanges. One of these lines is full and the other dashed, because one flange is on the near side and the other is on the far side of the web; this should correspond to the way the flanges are shown in the end view.

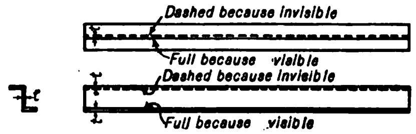

Fig. 40 (*b*). A *Z*-bar.

Top View. — The top flange is shown to scale between two full lines, one line showing also one face of the web; a dashed line is added to show the other face of the web, the web thickness being estimated as before. A fourth line is drawn to show the outer edge of the bottom flange, the space between it and the dashed line representing the flange width to scale. The position of the full and the dashed web lines should correspond to the other views.

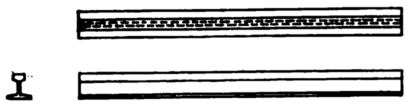

Fig. 40 (*c*). A Rail.

3. **A rail** is shown conventionally (Fig. 40 (*c*)) as follows:

End View. — The head, the web, and the flange are drawn from the dimensions given on page 317. On large scale drawings these are drawn to scale, and the curves are often shown. Ordinarily, however, straight lines are sufficient, the depth, the width and the thickness of the head, and the width of the flange being scaled.

Front View. — The head and the flange of the rail are each shown conventionally by two lines spaced approximately to show the mean thickness. The distance between the upper line of the head and the lower line of the flange is the depth of the rail to scale; each inner line is drawn so that, if extended, it would cut the sloping lines of the end view about midway between the web and the outer edges of the head or of the flange.

Top View. — The widths of the head and of the flange are each shown by two lines to scale. These lines are placed symmetrically about two dashed lines which represent the web, the web thickness being estimated.

4. **An eye bar** is shown in the main view to scale, from the dimensions given in the handbooks of the steel companies. The actual curves are drawn to scale. The edge and the end views appear as simple rectangles, although the curves are sometimes indicated by line shading, as shown in Fig. 40 (*d*).

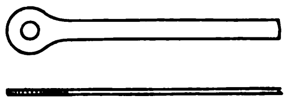

Fig. 40 (*d*). An Eye Bar.

5. **Lattice bars** are usually made from standard dies according to the billed size, as explained on page 45 : 2. For this reason it is unnecessary to show the bars on a drawing in detail. One or two bars are often shown on each sheet, being drawn from the dimensions given on page 315; other bars are indicated simply by center lines drawn from center to center of end rivets, as shown in Fig. 137. Often the intermediate bars and rivets of a group may be omitted (compare page 41 : 1), only one or two bars being shown or indicated at each end of the group. Dashed lines instead of full lines are sometimes used to indicate an independent system of lattice bars, as for example, when the bars on opposite sides of a member are shown in one view (Fig. 129).

6. **Shop rivets** and **holes for field rivets** are usually indicated in the views where they appear as circles according to the Osborne Code, as shown on page 304. An open circle of the diameter of the rivet *head* indicates the shop rivet, and a circle of the diameter of the rivet *hole*, but inked solidly, represents the field rivet. The diameter of the rivet hole is $\frac{1}{16}''$ greater than the diameter of the shank of the rivet (page 30 : 4), while the diameter of the head is as given on page 304. The drafts-

man can usually estimate the sizes of the circles closely enough when drawing to an ordinary scale; but he should be particular to show a contrast in size between the shop and the field rivets as an added safeguard in case he should neglect to fill in all the field rivets. It is well to remember that for a rivet of a given size the diameter of the circle for a shop rivet is approximately *one and one-half* times as large as the diameter of the circle for a field rivet.

After making a sample circle preparatory to drawing a large number of rivets, the beginner should measure the diameter to see if it is approximately to scale; in fact, this test is so quickly made that an experienced draftsman can often apply it to advantage, particularly when drawing to a small scale.

Other views of rivets and holes are shown only when an extra view may be avoided or the drawing may be made more clear thereby. In general the side or the sectional view of a hole for a field rivet is a rectangle filled in solid. A similar open rectangle represents the shank of a shop rivet to which are added semi-circular heads. See *G*20, Fig. 92. When shop rivets are countersunk, cross lines are drawn perpendicularly to each other and preferably at 45° with the rivet lines. See page 304. If countersunk on the *near* side (outside) the lines are only *outside* the circle, and if countersunk on the *far* side (inside), the lines are *inside*. Similarly the lines extend outside and inside if countersunk on both sides. To show this distinction in field rivets, auxiliary circles must be placed outside the others. Flattened rivets are indicated by lines which slope in one direction only, preferably at 45° with the rivet lines. The number of sloping lines shows the height of the rivet head in eighths of an inch, i.e., three lines for $\frac{3}{8}''$ and two lines for $\frac{1}{4}''$. Rivets flattened to $\frac{1}{8}''$ would be useless but they may be countersunk without being chipped after they are driven; they will then not project more than one-eighth inch. In order to insure the detection of countersunk or flattened rivets, notes are often added to supplement the code, especially when the rivets are in unusual places, as for example: "Rivets countersunk far side but not chipped," "Rivets countersunk and chipped both sides," or "Rivets flattened to $\frac{1}{4}''$ near side." (Figs. 129 and 133.) Such notes are omitted on shoe plates and column bases, or wherever rivets are usually countersunk, provided they are clearly indicated. Rivets should never be countersunk in metal thinner than that indicated in the table on page 304.

1. **All holes** for field rivets **are usually shown,** except in plate work or other work which requires a large number of field rivets, the position of which can be clearly indicated. All shop rivets should be shown in small connections and in all doubtful places, but some may be omitted when in large numbers if no ambiguity is likely to result. A short line crossing the rivet line is often used to indicate the center of a rivet. When shop rivets are dimensioned in a group (page 49 : 7), the rivets at the ends of the group are shown, but most of the intermediate ones may be omitted; it is well to indicate one space at each end of the group by means of a rivet or a cross line in order to emphasize the presence of intermediate rivets. On the pencil drawing enough rivets and holes should be shown to distinguish clearly between them when the tracing is made, without the necessity of further investigation. They may be made freehand in pencil, but they should be carefully made with a bow-pen or a "riveter" (page 60 : 3) on the finished drawing.

2. **Bolts** are occasionally drawn to scale from the actual dimensions (page 304), but ordinarily simply the holes for the bolts are indicated in the same way as holes for field rivets, whether the bolts are to be inserted in the shop or in the field. Shop rivets should never be shown where bolts are to be used. Bolts which are put in place permanently in the shop should be billed and noted on the drawing (page 53 : 3). If necessary to differentiate between shop and field bolts, small squares may be drawn around the solid circles for the shop bolts as illustrated in Fig. 140.

3. **Fillers** are used to fill the spaces between other surfaces and it is not usually necessary to draw any additional lines to represent them, since the lines would be coincident with lines already drawn. When clearance is allowed between the edges of the fillers and adjacent edges, additional lines (dashed if invisible) may be drawn, although these are sometimes omitted if the drawing is clearer without them. Round washers are used as fillers where there is only a single rivet, as at stitch rivets (page 69 : 4).

4. **Bent Plates.** — The true projections of bent plates and angles are usually simplified as shown in Figs. 143 and 149. The dimensions must

be shown accurately in the proper views, but most drawings would be complicated unnecessarily by an attempt to show all the edges. Often the drawing may be clearer and more easily interpreted if one view is drawn as if the plate or angle were not bent. The rivets and holes in inclined surfaces would appear as ellipses instead of circles (Fig. 78), but on account of the difficulty in making small ellipses this distinction is not always made on the drawing unless ambiguity would result from th use of circles.

1. **Other Materials.** — It is sometimes convenient to represent ma terials other than steel according to standard conventions. The mor common conventions for this class of work are shown in Fig. 42.

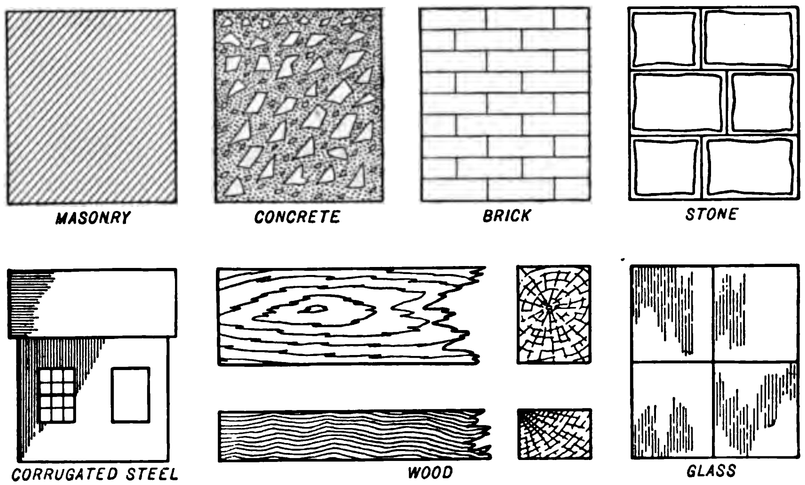

Fig. 42. Conventional Representations.

CHAPTER VII

STRUCTURAL DRAWINGS — THE CONVENTIONAL METHODS OF BILLING

SYNOPSIS: In Structural Drafting the term "billing" has two meanings, viz.: (1) the statement on a drawing of the number and the size of the component parts of a member; and (2) the preparation of a list or "bill" which is a summary of all the material used in the construction of one or more members, as an order bill (page 162 : 1), or a shop bill (page 167 : 1). In this chapter are explained the conventional methods of billing material on the drawings.

1. **Billing.** — The size or description of each component part of any member should be expressed or "billed" on the drawing near the principal view of that part. When two or more identical pieces occur together they may be billed together, but the number of pieces should indicate the number required at only one point of a single member. The object of billing is to show the commercial shape from which the part is to be cut, and the length of such shape required. The methods of billing must conform to the usual practice and indicate the commercial shapes in the same way that they are listed in the handbooks of the steel companies. The bill of material serves as the "name" of a piece, not only upon the drawing, the order bill, and the shop bill, but also upon the steel itself; it is painted upon the steel at the rolling mill as soon as the steel is cut to the ordered length, and it is used for identification until the piece is placed in its final position in the structure, or until it is assembled with other pieces to form a member which is subsequently identified by means of a shipping mark (page 80 : 6).

At the mill, the contract number as well as the bill of material is painted on the steel. If the material is recut in the shop to shorter lengths, the contract number and the revised bill are painted on each piece, along with the shipping mark and the assembling mark, if any (page 79 : 2). Similarly, the templets are marked to correspond. After a member is assembled and riveted it is painted before it is shipped; the contract number and the shipping mark are left uncovered or are repainted for use during shipment and erection.

2. Certain abbreviations and **conventional signs** are used in billing, as shown in the following paragraphs. The multiplication sign (×) is used simply to distinguish the different dimensions. It is pronounced "by"; thus $8 \times \frac{1}{2} \times 12'$-$0''$ is read "eight *by* one-half *by* twelve feet, no inches." Similarly, the sign (#) is used for "pounds," (#/ft.) for "pounds per foot" and (#/yd.) for "pounds per yard." In billing, the "per foot" and often the "per yard" are omitted. The size of the cross section is expressed in inches in *billing*, even if over one foot. The length is the extreme distance at right angles to the other figures; a length of $1'$-$0''$ or over is expressed in feet and inches, while a length less than one foot may be written in inches alone, unless it seems clearer to prefix the $0'$ thus: $0'$-$10''$. See the illustrations below. The conventional methods of billing the common structural shapes are as follows: —

3. **Plates**:

Number: Pl. (or Pls.): width: thickness: length
(pieces) (inches) (inches) (feet and inches)

Thus: 1 Pl. $8 \times \frac{1}{2} \times 10''$, or 14 Pls. $24 \times \frac{5}{8} \times 9'$-$10\frac{1}{4}''$.

Note that the width is *across* the grain, while the length is *along* the grain. See IX–(9), page 76 : 1. If possible, plates of even width, i.e., whole inches without fractions, should be chosen in order that stock sizes may be used without being recut.

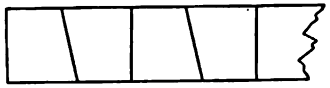

Fig. 44.

Usually the width is the smaller dimension, but this may be reversed in case the longer dimension can be made a stock width more conveniently; or in case material may be saved by cutting several plates as shown in Fig. 44.

When plates are used as fillers the Pl. or Pls. is replaced by Fill, or Fills. For other special abbreviations used in plate work see page 45 : 7. For the areas and the weights of plates see page 321.

1. **Angles:**

Number: L (or Ls): longer leg: shorter leg: thickness: length
(pieces) (inches) (inches) (inches) (feet and inches)

Thus: 1 L 6 × 4 × $\frac{1}{2}$ × 7$\frac{1}{2}$, or 12 Ls 3 × 2$\frac{1}{2}$ × $\frac{5}{16}$ × 10′-6″.

In the offices of some companies, the angle sign (L) is placed after the thickness, thus: 8-5 × 3$\frac{1}{2}$ × $\frac{5}{8}$ Ls-18′-3″. For special abbreviations used for angles see page 45 : 7. For the weights and the dimensions of angles see page 303.

2. **I-beams:**

Number: depth: I (or Is): weight: length
(pieces) (inches) (pounds per foot) (feet and inches)

Thus: one 12″ I 31$\frac{1}{2}$# 18′-0″, or 2′-15″ Is 42# 25′-7$\frac{1}{2}$″.

Note that if there is only *one* piece the number is spelled "one," but if more than one piece the number is expressed in figures. This obviates ambiguity between 1-8″ I, and 18″ I, etc., when poorly executed or indistinctly printed. For the weights and the dimensions of I-beams see pages 298 and 299.

3. **Channels:**

Number: depth: ⊔ (or ⊔s): weight: length
(pieces) (inches) (pounds per foot) (feet and inches)

Thus: one 12″ ⊔ 20$\frac{1}{2}$# 12′-10″, or 7-8″ ⊔s 11$\frac{1}{4}$# 15′-11$\frac{1}{2}$″.

Note that if there is only *one* piece the number is spelled "one," but if more than one piece the number is expressed in figures, as in I-beams. Note also that the channel sign (⊔) is made preferably with the web horizontal to diminish the liability of confusion with the I-beam (I) if poorly made. A special interpretation is given to different ways of making this sign on floor plans. See page 157 : 2. For the weights and the dimensions of channels see pages 300 and 301.

4. **Rods:**

Number: diameter or side: ○ or □: rod (or rods): length
(pieces) (inches) (feet and inches)

Thus: 1-$\frac{3}{4}$″○ rod 12′-0″, or 10-1″□ rods 11′-0$\frac{3}{4}$″.

Note that a circle (○) is used for round rods and a square (□) for square rods.

5. **Tees:**

Number : T (or Ts) : width of flange: length of stem: weight: length
(pieces) (inches) (inches) (pounds per foot) (feet and inches)

Thus: 1 T 3 × 3$\frac{1}{2}$ × 8.6# × 12′-11″, or 3 Ts 4 × 3 × 9.3# × 13′-2″.

Note the distinction between a 3 × 3$\frac{1}{2}$ (3″ flange) and a 3$\frac{1}{2}$ × 3 (3″ stem), etc.

6. **Z-bars:**

Number: Z (or Zs): depth: width of flanges: thickness: length
(pieces) (inches) (inches) (inches) (feet and inches)

Thus: 1 Z 6 × 3$\frac{1}{2}$ × $\frac{9}{16}$ × 8′-3$\frac{3}{4}$″, or 4 Zs 3$\frac{1}{16}$ × 2$\frac{3}{4}$ × $\frac{5}{16}$ × 17′-0″.

7. **Rails:**

Number: Rail (or Rails): weight: standard: length
(pieces) (pounds per yard) (feet and inches)

Thus: 1 Rail 85 #/yd. A.S.C.E. 20′-0″, or 16 Rails 100 #/yd. A.R.E.A. 33′-0″. Note that the weights of rails are given in pounds per *yard*, instead of pounds per foot as in the case of other shapes; for this reason it is preferable to write the weight as above, although the (#/yd.) is often written simply (#). The standards of the American Society of Civil Engineers, the American Railway Engineering Association, and the American Railway Association are in common use. The usual

lengths are 30 ft. for rails up to 60 #/yd. and 33 ft. for the heavier rails. For the dimensions and the properties of rails see page 317.

1. **Eye bars:**

Number : eye bar (or eye bars) : width of main bar : thickness : length c. to c. of holes
(pieces) (inches) (inches) (feet and inches)

Thus: 1 eye bar 14 × 1 × 15′-0″ c. to c. of holes, or 2 eye bars 8 × $\frac{3}{4}$ × 12′-0″ c. to c. of holes.

Note that the lengths of eye bars are given from center to center of pin holes instead of the extreme length.

Eye bars are connected by means of pins. The distances from center to center of pins are calculated, and the eye bars should be made to correspond. The heads of the eye bars are upset while hot, and the holes are then sub-punched, i.e., punched to a smaller diameter than the required size. Two boring machines are carefully set so that the finished holes at both ends of a bar can be bored simultaneously at the exact specified distance apart. The over-all length is therefore relatively unimportant.

2. **Lattice bars:**

Number: Latt. bars: width: thickness: length c. to c. of holes
(pieces) (inches) (inches) (feet and inches)

Thus: 22 Latt. bars $2\frac{1}{4}$ × $\frac{1}{4}$ × 1′-$2\frac{3}{8}$″ c. to c.

Note that the lengths of lattice bars are given from center to center of rivet holes. Since this is not in accord with the method of billing other material, the lengths should always be followed by "c. to c."

The distance center to center of holes is much more important than the extreme length in order to insure an accurate matching of the holes. Furthermore, this distance is used in setting the adjustable stop in a special lattice bar punch. This machine cuts out the material between the curved ends of two bars and punches the holes in these ends simultaneously according to standard dies, as indicated on page 315. As the bar is advanced to the adjustable stop and punched again, a complete lattice bar is made at a single stroke.

3. **Washers:**

Number: washer (or washers): diameter: thickness
(pieces) (inches) (inches)

Thus: 4 washers $2\frac{1}{4}$ × $\frac{1}{4}$.

4. **Rivets:**

Number: diameter: rivet (or rivets): length
(pieces) (inches) (inches)

Thus: 450-$\frac{3}{4}$″ rivets $1\frac{1}{2}$″ long.

If rivets have countersunk heads the fact should be stated. The diameter of a rivet is the diameter of its shank (page 30 : 4). The length of a button head rivet is from the underside of the head to the end. The length of a countersunk rivet is the extreme length, including the thickness of the head (page 304). The number and the lengths of rivets are seldom specified except on the rivet lists from which the field rivets are made and shipped. The size of shop rivets is given on structural drawings thus: $\frac{3}{4}$″ rivets.

5. **Bolts:**

Number: bolt (or bolts): diameter: length
(pieces) (inches) (feet and inches)

Thus: 2 bolts $\frac{1}{2}$ × 2, or 4 bolts 1″ × 1′-3″ lg.

If a bolt has a countersunk head the fact should be stated. The diameter of a bolt is the diameter of its shank (page 304). The length of an ordinary bolt is from the underside of the head to the end. The length of a countersunk bolt is the extreme length, including the thickness of the head.

6. **Holes:**

Diameter: hole (or holes)
(inches)

Thus: $1\frac{3}{8}$″ holes.

Rivet holes are made $\frac{1}{16}$″ larger than the diameter of the rivets to be inserted. (Why? See page 30 : 4.) Most bolt holes in structural work are made $\frac{1}{16}$″ larger than the corresponding bolts, except holes for anchor bolts (pages 73 : 3 and 106 : 5).

7. **Special Abbreviations.**—Adjectives may be used before the abbreviations for plates and angles to aid in identification, as for example:—

Bear. Pl. for bearing plate,
Bent Pl. for bent plate,
Cov. Pl. for cover plate,
Fl. Pl. for floor plate,
Reinf. Pl. for reinforcing plate,
Spl. Pl. for splice plate,
Web. Pl. or Web for web plate,
Flge. L. for flange angle.
Spl. L. for splice angle,
Stiff. L. or Stiff for stiffening angle.

CHAPTER VIII

STRUCTURAL DRAWINGS — THE DIMENSIONS

SYNOPSIS: Structural steel cannot be trimmed and fitted during erection as wood is cut in carpentry, but all parts of a structure must be made to fit the first time. Consequently, more elaborate precautions must be taken, first to insure the correctness of all dimensions, and second to make sure that the dimensions are so expressed on the drawing that they cannot be misunderstood. In few kinds of work is there a more exacting system of painstaking precautions and checks than in the work in a modern structural drafting room. The drawings must be issued absolutely free from mistakes, as far as possible, for each uncorrected mistake, unless detected in the shop, may cause the loss of a large sum of money. (For example, the use of 30′-0″ instead of 30′-6″ for the lengths of 1200 I-beams once caused a large loss.) In this chapter are given rules and suggestions for dimensioning a structural drawing, including the use of the dimension lines, the arrow heads, and the figures.

1. In this book a distinction is made between the terms "**dimension**" and "**size.**" The former implies the use of a dimension line upon which the dimension is written, whereas the "size" applies to the figures which are used in billing (Chapter VII, page 43), and may refer to the depth, the width, the thickness, the weight, the length, or to various combinations of them.

2. **The dimensions** form one of the principal parts of the working drawing. The shopmen and the draftsmen who use the finished drawings are never permitted to scale them, but must always use the figured dimensions. It is therefore important that all dimensions should be accurate, and the extent of each dimension should be apparent. A dimension is worthless unless it is of some use, unless it is perfectly legible, and unless there is no ambiguity regarding the two points between which it is intended.

3. Dimensions should indicate the **actual measurements** of the piece represented, regardless of the scale used in the drawing.

4. The dimensions should be **placed upon the drawing as soon as determined,** while fresh in mind. The dimensions should usually be determined in the order of importance, the main dimensions first, and those for the details last. The main dimensions for a drawing are generally determined from the erection diagrams or the design sheets, while the dimensions for the details are found in the tables of this book, in the handbooks or the standards of the different steel or structural companies, or else they are supplied by the draftsman.

5. **Position.** — Each dimension should be placed upon the drawing in such a manner that it will be of most use to those who read the drawing. The principal dimension should be made conspicuous by being placed upon a dimension line which intersects the fewest possible number of other lines. Thus the longest dimensions, such as over-all lengths and extreme depths, should be placed on dimensions lines which are farthest from the view to which these dimensions apply (usually the front view), and the shorter dimensions, such as those for rivet spacing and

other subdivisions, should be on dimension lines nearest the view. In this way the lines for long dimensions are not crossed by the perpendicular projection lines which mark the ends of shorter dimensions. When there are several dimension lines close together, it is often desirable to make the figures of the principal dimension larger and bolder than the rest.

1. **Dimension lines** should be continuous black lines, as fine as practicable without appearing ragged (page 37 : 1). They should be drawn parallel to the measurements to be dimensioned, and should extend between projection lines drawn at right angles to the dimension lines to indicate the distances intended.

2. Dimension lines should usually be placed **outside of the view** dimensioned, and preferably *between* the views if more than one. At times, however, a few dimensions may be placed in a clear part of the view itself if space or clearness is gained thereby.

3. Dimension lines are usually placed about $\frac{1}{4}''$ apart, although this may be reduced to $\frac{3}{16}''$ or increased to $\frac{5}{16}''$, depending upon the available space. The distance from the view to the nearest dimension line is made twice the **space between dimension lines,** unless it must be increased to allow for projecting details, or for other shorter dimension lines.

4. **Arrow heads** should be placed on the dimension lines to indicate the extent of the dimensions. They should be made definite, otherwise the dimensions are useless. The size and the style of arrow heads depend upon their location. For main overall dimensions they may be made large and bold, as in Fig. 47 (*a*). Flat curved arrow heads appear more graceful and may be used wherever there can be no ambiguity, Fig. 47 (*b*), but when a dimension extends to one of two lines which are close together, the arrow heads should be made short and wide, so that the vertex is distinct. See Fig. 47 (*c*). They should not be made large when close together. Arrow heads generally point away from the dimension figures, but in narrow spaces they may be reversed. See Fig. 47 (*d*). A dimension extends from the vertex of one arrow head to the vertex of the first arrow head in the opposite direction. Usually only one dimension is placed between these arrow heads and this should be placed midway between them (except extension figures, page 86 : 5).

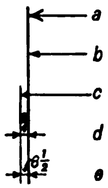

Fig. 47.

5. **Dimension figures** should be made large and distinct. Words if illegible can sometimes be guessed, but figures never. Each digit should be carefully drawn or printed, not hastily made as in computation. Beginners have a tendency to make the figures too small or with too thin lines; they appear still thinner if not entirely illegible on the blueprint. Figures made with a crow-quill pen, for example, are altogether too light. Simple figures should be made; curves should be avoided where straight lines can be used, thus:

1 not *1*, *2* not *2*, *3* not *3*, *4* not *4*, *5* not *5*.

6. Dimension figures should usually be placed just **above the dimension lines.** This practice differs from the system employed in machine drawing, where the figures are placed in breaks in the lines. In structural drafting much time is saved by drawing the dimension lines the full length, and then subdividing them into the required number of parts at will, particularly if this number is large. Dimensions should never be placed upon lines which are used for other purposes, as for example, main lines of the drawing, center lines, rivet lines, or projection lines for dimensions at right angles to the ones in question.

7. **Fractions** should be made with the lines horizontal, *never* inclined. Thus, not only is space economized, and the general appearance of the drawing improved, but many serious mistakes are obviated; for example, *1 1/16* might be either eleven-sixteenths or one and one-sixteenth. The figures of fractions should be nearly as large as the whole numbers, instead of one-half as high. If a figure of a whole number should be of a certain size to be clearly legible, it is equally important that each figure of a fraction should be of this size.

8. **Mistakes.**—If a wrong figure is made it should be **erased** entirely and a correct figure should be made in its place. A correct figure should never be superimposed on an incorrect figure, even if made heavier, for the man in the shop may not know which is correct; neither should the first figure be crossed out and a new one written above it.

This is contrary to the usual practice in keeping field notes in surveying in which a line is drawn through an incorrect measurement and the correct value written just

above. In surveying this indicates to the man who plots the notes that the measurement was repeated, which fact may prove of value when the notes are adjusted. In structural work the drawings are used only by men who are interested in the correct values, and nothing can be gained by exposing the draftsman's mistakes.

1. When the **space between the arrow heads is so limited** that the dimension figures cannot be placed on the drawing in the usual manner, it is frequently possible to compress the figures laterally without reducing their height, thus making little apparent alteration, as shown in Fig. 47 (*d*). In case this method cannot be adopted the figures can be placed either below the line between the arrow heads, or else a little to one side with an arrow leading to the corresponding space, as shown in Fig. 47 (*e*). In all cases the figures should be made parallel to the dimension line.

2. All figures and notes should be arranged so that they can be **read from the bottom edge or from the right-hand edge** of the sheet, or from a position between the two; i.e., the lettering should read from the left toward the right or from the bottom toward the top. Draftsmen soon become so familiar with figures and notes in these positions that they can read them all without turning the sheet. On lines which slope upward to the left and downward to the right, particularly on lines which are more nearly vertical than horizontal, the direction of the lettering is not so rigidly determined by a general rule but is left to the judgment of the draftsman. If the sloping dimensions of a member are used in conjunction with vertical dimensions more than with horizontal ones, it is often more convenient to have the figures read from the bottom toward the top; otherwise, it is better to print them from the left toward the right so that they can be read from the bottom of the sheet without turning the drawing.

3. All dimensions should be expressed in **feet and inches** and fractions of inches. The fractions should be reduced to the simplest form thus: $\frac{3}{4}$ *not* $\frac{12}{16}$. Usually no dimension should be used in structural drafting which is not a multiple of one-sixteenth of an inch; in fact it is desirable to avoid fractions in sixteenths wherever possible, as explained on page 34 : 6.

4. **Decimals** should be avoided in dimensions. Decimals from the handbooks should be converted into fractions, either by means of a table (page 333) or mentally. Approximately, $0.06 = \frac{1}{16}$, and this gives the correct result if referred to the nearest quarter of an inch, as for example: — 0.81 is 0.06 more than 0.75, hence $\frac{1}{16} + \frac{3}{4} = \frac{13}{16}$; and 0.19 is 0.06 less than 0.25, hence $\frac{1}{4} - \frac{1}{16} = \frac{3}{16}$. If a web thickness given in decimal form should fall midway between sixteenths, the higher sixteenth is usually chosen in dimensioning to allow for "packing," since paint, scale, bends, etc., do not permit the surfaces of two pieces to be brought into perfect contact. The tables at the end of this book express the web thicknesses in both decimal and fractional forms.

5. The abbreviations (ft.) and (ins.) are not used in dimensioning. A single accent mark (′) represents feet, and a double accent (″) inches. Dimensions less than one foot are usually expressed in inches alone, but to avoid ambiguity dimensions of one foot or over (even if even feet) should always show both feet and inches with hyphens between.

Note: The width of a plate is given in inches when *billed* (page 43 : 3), whether more or less than 12″, but when given as a dimension with a dimension line it is expressed in feet and inches if 1′-0″ or over. Hence it is no exception to the general rule.

The inch marks (″) of dimensions less than one foot may often be omitted, provided there can be no doubt as to the meaning; but the inch marks should be used whenever the drawing can be made more clear thereby. The correct **method of writing dimension figures** can best be shown by examples, as follows: —

Correct		Incorrect
$\frac{1}{2}$	not	$0\frac{1}{2}$
2″, or $6\frac{1}{2}$	not	0′-2″, or 0′-$6\frac{1}{2}$″
1′-0″	not	1′ or 12″
2′-0″	not	2′
3′-$4\frac{1}{4}$″	not	3′-$04\frac{1}{4}$″
4′-$0\frac{3}{4}$″	not	4′-$\frac{3}{4}$″, or 4′-$00\frac{3}{4}$″

6. **Recurring Dimensions.** — Ditto marks (") should never be used in place of dimensions. The use of arrows leading from one figure to two or more spaces should be avoided. Like dimensions should be repeated at every occurrence, unless grouped as explained on page 49 : 6.

1. A dimension which is clearly given in one view **should not be repeated** in another, for it usually complicates the drawing unnecessarily, and causes trouble if one is changed.

2. **Rivets and holes** should be located by dimensions which extend to their centers. They should be dimensioned in the views which show them as circles, although exceptions to this rule are occasionally allowed in order to dispense with additional views (page 91 : 4). "Staggered rivets," i.e., rivets which alternate on two lines, should be dimensioned, not diagonally, but parallel to the rivet lines as if the rivets were on a single line. See Fig. 49.

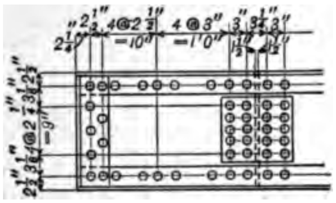

Fig. 49.

3. **The gages** of all structural shapes should be given in all cases (except latticed members, page 136 : 3). The standard gages given in the tables should be used, except in special cases which are noted elsewhere, as for example on pages 83 : 6, 106 : 3, 132 : 1, 132 : 3, 136 : 1, and 139 : 3.

4. The majority of **edge distances** are omitted unless they are important. For example, a dimension should be given in order to limit an edge distance to provide ample clearance for a connecting part (page 72 : 4), or to tie a group of rivets to an important edge to which a main dimension extends (Fig. 148). But if a connection plate or angle, or a web member of a truss, is shown without edge distances, the shopmen will make the distances on opposite edges equal; with this understanding many dimensions may be omitted.

5. **A line of rivet spacing** should be confined to dimensions from center to center of rivets and holes, except at the ends where it is frequently necessary to give the distance from the first rivet to the end of the member or to some other definite point. Intermediate edge distances and gages should never be given on the same line, because they are not used at the same time; they should be shown as separate dimensions, usually between the line of rivet spacing and the corresponding view of the member. See Fig. 49.

6. Dimensions should never be given to the **edges of the flanges** of structural shapes, but always to the backs of channels and angles, to the center lines of I-beams, and similarly for other shapes. The back of an angle or a channel is a well-defined line, but the outside edges are not so well defined. More important than this, however, is the fact that the lengths of legs and the widths of flanges are frequently different from what they are supposed to be. As a rule, such variation is of no consequence in structural work, but it should be considered, particularly in the case of thick angles. See page 25 : 1. For illustration, suppose a girder is composed of a $\frac{3}{8}''$ web plate and 6 × 4 angles. The total width of each flange is theoretically 1'-0$\frac{3}{8}''$, but if the angles should "overrun," as they are likely to do, the width would be more. Ordinarily this increase would not matter and for this reason the dimension should not be given; in case the extreme width is limited by the clear distance between column flanges or for other reasons, the dimensions should be given with the understanding that in case of overrun, the outside edges of the angles would have to be planed — a process too expensive to be used unnecessarily.

It is unnecessary to give the widths of flanges or the dimensions of fillets and other curves which apply to the manufacture of the steel rather than to the fabrication. It should be borne in mind that the drawings are to be used for the purpose of putting standard shapes together, and therefore many dimensions may be superfluous unless they are directly related to shearing, bending, punching, riveting or similar processes.

When an angle with unequal legs is represented in only one view it is often necessary to indicate which leg is shown. This may be noted as for example "3″ leg," but more frequently the length of the leg is put on in the form of a dimension with the understanding that it is no more exact than the billed length and that the angle need not be cut in case of overrun.

7. When three or more spaces are numerically equal and serve similar purposes, they may be **dimensioned in a group,** thus: — 5 @ 6 = 2'-6″, or 4 @ 1'-2″ = 4'-8″, or as some companies prefer, 5 of 6 = 2'-6″. If the rivets are staggered (page 49 : 2), the spaces are given just as if the rivets were on the same line, although the abbreviation "alt. spa." for "alternate spaces" is sometimes added, thus: — 5 alt. spa. @ 6 = 2'-6″. Edge distances and gages should not be combined with distances from center to center of rivets and holes in this method, but should have the dimensions repeated, even though identical. In the same way it may be preferable to separate one or more of the spaces from the group if they serve purposes in addition to the spacing of

rivets along the same line as the rest of the group, as for example, the $2\frac{1}{2}$ and 3 inch spaces in Fig. 49 which locate rivets in the stiffening angles and splice plates.

1. When a long line of shop rivets and holes for field rivets are dimensioned in a single group, and the holes occur at such intervals that spaces must be counted to determine their location, **a supplementary dimension line** may be added for the holes only, as in the top flange, Fig. 103.

2. If two or more lines of dimensions extend between the same points. **the sum of the dimensions** in each line should be the same as in the others. If a line of dimensions extends practically the whole length of a member it is well to complete it by adding one or more dimensions to afford a check with the over-all length. This is of special benefit to the templet maker, who would otherwise have to procure his own total for a check.

Checking Subdivisions: Never complete a line of dimensions between two points without adding them to see if the sum equals the proper distance between those points. Neglect of this precaution is a source of much trouble.

3. Dimensions should be placed upon the drawing in such a manner that the **shopmen will not be compelled to add or subtract** dimensions in order to obtain the figures they need. For example, one hole in the connection angle on the flange face of column *C*5 Fig. 137, is tied to one rivet in the other leg (or else each could be tied to the same end of the angle); otherwise, the templet maker could not make the templet for the angle without finding the total distances from the rivet and from the hole to the bottom of the column and then subtracting these two sums. Similarly, the holes in the connection plate *pb* near the center of this same column are dimensioned independently of the rivets; a line is drawn, however, to show that the top holes and rivets are opposite.

4. No rivets or holes should be located by more than **one method of dimensioning.** They may be determined either by dimensions at right angles to each other (rectangular coördinates), or by "slopes and distances" (polar coördinates), the slopes of the rivet lines being indicated in the usual manner (page 50 : 7) and the distances along these lines being given. A combination of these two methods is not only unnecessary, but is liable to cause difficulty in the shop, for the points that coincide on a small layout in the drafting room might fall noticeably apart on the full-sized templet. In order to avoid ambiguity on the drawing in case a line which locates one rivet or hole happens to pass through another rivet or hole which is located in another way, the line should be made to pass around the second rivet or hole by means of an arc of a circle, as shown in Fig. 50 (*a*). Whether the drawing is made accurately to scale or whether the line would actually pass through the center or not is immaterial, the object of the arc being to show clearly that the line is independent of the second rivet or hole.

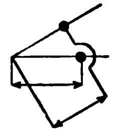

Fig. 50. (*a*)

5. For **field connections** — connections of different members in the field — the dimensions on the drawing of one member should be given in exactly the same way as the corresponding dimensions on the drawing of the other member so that each person who uses the drawings may see at a glance that these dimensions do correspond.

6. The rivets in the ends of the **lattice bars** of a single system of latticing are dimensioned in groups much as staggered rivets are dimensioned (page 49 : 2); each space is measured parallel to the axis of the member latticed, from the center of the rivet at one end of a bar to the center of the rivet at the other end. See Fig. 137. The method of dimensioning a double system of latticing is the same as for a single system, the rivet at the intersection of each pair of bars being shown but not dimensioned. See Fig. 124. For lattice bars with two rivets at each end, see Fig. 127.

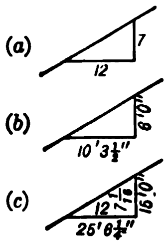

Fig. 50 (*b*)

7. **The slope of a line,** i.e., its "bevel," with reference to another line is given, as shown in (*a*) Fig. 50 (*b*), by indicating dimensions on two mutually perpendicular sides of a right triangle; the hypothenuse of this triangle is either coincident with or parallel to the line whose direction is to be given, and the other sides are respectively parallel and perpendicular to the reference line. The dimension on the longer perpendicular side is always 12″, while the corresponding dimension (in inches and fractions) on the shorter side must be calculated. This shorter dimension is usually obtained directly from dimensions between working points, as explained on page 76 : 2, without the necessity of

knowing the actual value of the angle. Angles are almost never given in degrees and minutes on structural drawings but the corresponding slopes are used.

A workman in the shop usually has no such means as an accurate protractor for laying off angles, but he must work with a two-foot rule, a steel tape, or other device for laying off linear measurements.

Parts of trusses or bracing systems are often laid out on the floor of the templet shop from the main dimensions instead of on the bench, provided that the extreme dimensions do not exceed 30 or 40 feet. If the dimensions between working points are not clearly shown on the drawing they may be given on the triangle without being reduced to a base of 12″, as shown in (*b*) Fig. 50 (*b*). The two schemes may be combined as in (*c*) Fig. 50 (*b*) in order to show the dimensions between working points for the convenience of the checker, and the corresponding reduced dimensions for use in the shop. This method may be used when a draftsman is in doubt as to whether the work will be laid out on the floor or on the bench.

CHAPTER IX

STRUCTURAL DRAWINGS — THE NOTES, THE TITLE, AND THE BORDER

Synopsis: Structural drawings are made as self-explanatory as possible without supplementary notes, although notes can often be used to advantage. Some of the more common notes are discussed in this chapter, and suggestions are given for making titles and borders.

THE NOTES

1. Drawings should be made complete and clear without the use of notes whenever practicable, but unless the drawing is perfectly clear, short and concise notes should be added wherever necessary. Care should be taken to make each note explicit and easily understood.

2. All **general notes** which apply to the whole drawing should be placed near the title, usually to the left, as in Fig. 98. Examples of the more common general notes are those which give the sizes of rivets, holes, and washers, the maximum pitch of rivets, and information regarding specifications, paint, inspection, and erection.

3. All **other notes** should be placed in clear spaces near those parts of the drawing to which they apply. If a note is too long for the available space, it may be placed to one side with a reference to it at the proper point, as for example, "See note." If more than one such case occurs on one sheet, the notes may be numbered or lettered.

4. If the **rivets and holes** for any drawing are not all of the same size, the general note (see above) should be modified thus: "Rivets $\frac{3}{4}$" except as noted," or "Holes $\frac{13}{16}$" unless noted" (Fig. 104). All rivets or holes of sizes other than those specified in the general note should be clearly noted. Such exceptional rivets or holes may be distinguished on the drawing in one of several ways, thus: (a) by placing the note at one end of that dimension line which locates all the rivets or holes in question but no others, as in *P*6, Fig. 90; (b) by drawing supplementary lines with arrows to indicate the desired rivets or holes, as in *ME*, Fig. 117, or *F*10, Fig. 147; (c) by drawing arrows from the note to those rivet lines which pass through all the rivets or holes noted but no others, as in the bottom view, Fig. 135; (d) by encircling the desired rivets or holes by a freehand loop or ring which in turn is connected to the note, as in the top view, Fig. 135; this ring should not include any dimensions or notes (page 181 : 1); (e) by noting all the rivets or holes in one view by a special note near the view as in *B*16, Fig. 92, or by a general note near the title as in Fig. 99. In order to attract attention to a change of size, the American Bridge Company has adopted the use of a heavy triangle as illustrated in most of the figures referred to above. Countersunk and flattened rivets are often noted as explained on page 40 : 6.

5. **An identification mark** is assigned to each member which is to be shipped separately. If more than one drawing appears on a sheet the identification mark should be placed conspicuously near the drawing of the corresponding member. For further description of such "Shipping Marks," see page 80 : 6.

6. **The number of pieces** to be made from each drawing should be clearly stated, either under the detail, as in beam work (Fig. 87), or at the right of the sheet above the title (Fig. 100). When more than one or two members are shown on one sheet a tabular "Required List" is made, as shown in Fig. 147.

1. **Reference to other drawings** may be made in order to save the repetition of details. Usually reference is not made to another sheet unless a system of assembling marks is used, as discussed more fully on page 79 : 2. The size of all material should be given on each sheet, and also the spacing of all holes for field connections to facilitate comparison with the drawings of connecting members. The outline of all material should be completely shown in at least one view to prevent the shopmen from overlooking a whole connection or other detail. The shop rivets and the dimensions which locate the shop rivets and cuts may be referred to another detail, provided the latter is complete in every respect without further reference to a third detail.

2. **Loose Pieces Bolted.**—Small pieces, such as splice plates, fillers, or connection angles, which cannot be riveted in the shop without complicating the field work, may either be shipped separately, or else bolted for shipment to one of the members to which they are eventually to be attached. The latter method is to be recommended wherever feasible, for the number of items shipped is reduced, and the pieces are available at once when needed for erection. Pieces which are bolted for shipment are fastened temporarily in or near their final positions by means of two or more bolts each. On the drawing, a note should be placed immediately following the billed size of any piece so bolted, thus: "Bolt for shipment," or "Bolt to ship." See Fig. 125. It is well to give each piece a separate mark for identification in case it becomes detached unless the nature of the piece is such that its proper position can be readily determined.

The temporary bolts used are of odd lengths as picked up in the shop; sufficient washers are added to facilitate tightening and removing the bolts.

To avoid handling loose pieces in the field it is often feasible to make them enough longer so that they can be riveted in the shop independently of the field connections (Fig. 160); this should never be done, however, if the erection is rendered more difficult thereby.

3. In case pieces are to remain bolted in the completed structure without having the bolts replaced by rivets in the field, all the holes should be filled with **permanent bolts** of the proper size. The bolts should be billed on the drawing in the usual way (page 45 : 5), directly under the billed size of the piece bolted. See Fig. 133. A note, "Bolt complete" may be added to insure the use of a full number of bolts of the proper length, instead of a few temporary bolts (see above). The bolt heads are sometimes drawn but ordinarily, for the sake of simplicity, only the holes are indicated (page 41 : 2).

4. **Different Members Combined.**—When several members differ from each other slightly, i.e., in minor details, one drawing may be made to represent all of these members and the differences may be stated in notes. This should not be done, however, unless the differences can be made clear and definite in notes which are brief and comparatively few in number; otherwise, drawings become so complicated that more time is lost by the shopmen than is saved by the draftsmen. This is often due to the fact that additional connections are discovered, or the design is changed, after the drawing is started. It is essential, therefore, to plan any combination very carefully before the drawing is commenced, using a sketch upon which are indicated the various connections. The notes regarding special connections follow the corresponding billed material, if any; otherwise, the holes may be noted as indicated on page 52 : 4. If there are more than a few simple differences, it is better either to draw an additional view (Fig. 137), or to make another drawing. In the latter case it may be possible to show on one drawing simply the outlines of all parts which are like those on the other, and to draw in detail only those parts which are different (Fig. 117). In order to shorten notes, when several different members are represented by the same drawing, one member may be referred to another after the shipping mark, as for example:—"*C*12, same as *C*11 except as noted," it being understood that every note which applies to *C*11 applies also to *C*12. The mark "*C*12" will not appear in any of the notes, therefore, except where it differs from *C*11. See also page 81: 3.

5. **All notes should be made positive,** except in rare instances. This precludes the use of the word "omit." For example, if several members are represented by one sketch, and some of the details are on part of the members only, as the holes for rods in *MN*, Fig. 117, it is better to note that the holes are "in *MN*1, and *MN*2" rather than that they are "omitted in *MN*3" It is much more important to all concerned in making a member to know which details are to be placed on that member rather than to know which are not.

THE TITLE

1. A title is placed in the **lower right-hand corner** of each full-sized sheet, for convenient reference and for use in classification. This uniform position enables a person to look through a pile of drawings for a particular sheet by merely turning back the corners. The title is composed of two parts, one of which refers to the specific drawing, the other to the company which makes the drawing.

2. **The first part** of the title contains the names of the members on the sheet, or the part of the structure shown; also the name of the structure, the name of the customer, and the place where the structure is to be situated. This part of the title is usually made of freehand letters about $\frac{3}{16}''$ high, of simple Gothic style, and all capitals. Elaborate titles should be avoided, since they add to the expense, and are not in harmony with the rest of the drawing. Some companies maintain small presses for printing the titles of the larger contracts.

3. **The second part** of the title is more general, and is usually printed by means of a press or placed on the drawing by means of a rubber or metal stamp. If the drawing is made in the drafting room of a structural company, the title contains the name of the company, and the name of the plant which fabricates the work; also blanks for the initials of the man in charge of the contract, of the detailer, of the tracer, and of the checker, together with the date of each signature. Additional blanks are often left for the signature of the man who makes a field check or the engineer who approves the drawing. Below these, and at the extreme bottom of the sheet next to the border, are placed the contract number and the sheet number. The contract number should be made bold and conspicuous and in a uniform position. A single letter "C" can be made more prominent than either "Cont. No." or "Contract Number" and although not as significant it has been found convenient for reference. If desired, the line for the number may be placed opposite the middle of the letter, leaving room for the sheet number below, as shown in many of the titles on the drawings of this book. Titles made in the offices of consulting engineers or in the structural departments of railroad or other companies are usually somewhat simpler than those made in the offices of structural companies; in general they are similar, but they have comparatively few signatures.

4. **The smaller drawings** and lists are usually made on printed forms which contain the name of the company with blanks for the plant, the structure, the nature of the drawing, the initials of the detailer and the checker, with dates, and the contract and sheet numbers as shown in Fig. 85.

5. **Sheet Numbers.** — Drawings should be numbered in different series in order to facilitate the reference to a sheet of a given number. For example, the main drawings (24″ × 36″) of a contract may bear simply numbers, but other sheets should have a prefixed letter to distinguish them, thus: E for erection diagrams, B for beam details and other small drawings, C for combination sheets, S for shop bills, R for shipping bills, SR for combined shop and shipping bills, EF for miscellaneous lists, etc.

THE BORDER

6. A simple border should be drawn on each full-sized sheet to form the boundary, to keep the sheets of uniform size, and to improve the appearance. Ornate borders should be avoided. Corners should be simply rectangular. Borders are not drawn on the smaller printed forms.

7. An effective border is composed of a heavy line with a light line $\frac{1}{2}''$ outside. The nominal dimensions of the sheet, as for example 24″ × 36″, indicate the size of the finished tracing, and a rectangle of this size should be penciled to serve as a guide in trimming the cloth. The fine line is drawn $\frac{1}{2}''$ inside of this to indicate where the blueprints are usually trimmed, and the heavy line is drawn $\frac{1}{2}''$ inside the fine line to form the border on the prints. This makes the dimensions inside of the border each two inches less than the nominal size of the sheet, as for example, 22″ × 34″ for a 24″ × 36″ sheet. If smaller sheets are adopted, as for example 15″ × 22″ for student use, the fine line of the border had better be omitted, the heavy line only being used, $\frac{1}{2}''$ inside the edge of the sheet. In this case the tracing cloth and the blueprints should both be cut the same size as the sheet of paper. The heavy line should not be over $\frac{1}{32}''$ wide nor wider than can be drawn with a single stroke. See page 59:3.

CHAPTER X

INKING AND TRACING

SYNOPSIS: Every draftsman should cultivate the ability to ink a drawing well. Some men are naturally more skillful than others, but any draftsman can develop skill with practice, if he follows with reasonable care the common rules for inking. Such rules and suggestions for inking are given in this chapter.

1. **Structural drawings** are made primarily for use in the shop, and therefore accuracy, speed, and utility are of much more importance than appearance. On the other hand, it is highly desirable that a drawing be neat, well arranged, and well executed, although it is not so important in this work as in map or architectural work. A good looking drawing not only adds to the prestige of the draftsman, but also gives to all who use it greater confidence in its accuracy. The appearance of a drawing is determined largely by the arrangement on the sheet, and also by the skill of the person who inks it. There are many practical rules for inking and tracing with which every draftsman should be familiar, whether he is a skilled tracer or not; by their adoption he will be enabled to make drawings much more definite and useful than he could otherwise, and he will gradually develop skill along the proper lines.

2. **Three methods of making drawings** are commonly used. A drawing may be made on paper and then inked; it may be made on paper and then traced in ink upon tracing cloth; or it may be made directly in ink on tracing cloth. The rules for inking are practically the same in all three methods.

3. **The Care of Tracing Cloth.** — Tracing cloth should never be folded or creased, for permanent cracks would result. It should not be handled with moist hands, and water should not be allowed to come in contact with it, for not only would the surface be thus spoiled for inking, but it would also be rendered opaque so that white spots would show on each print taken from the tracing.

4. **The dull or unglazed side** of the tracing cloth is used for structural drawings for three reasons, viz.: (1) the dull side is the only side upon which pencil marks can be readily made or erased; not only is this of advantage at times to the tracer and to the detailer but it permits the checker to note corrections in pencil directly on the drawing (page 181 : 1); moreover, if a part or the whole of a drawing is to be made in pencil directly on the tracing cloth (page 65 : 3) it must necessarily be made on the dull side; (2) extensive erasures may be made with less apparent unjury to the cloth than on the glazed side, especially if several erasures are made in one place; and (3) the tracings may be more easily handled and filed because they are less liable to curl.

5. Before the cloth is stretched, **the selvage edges** should be torn off. These edges are woven with the threads closer together than in the body of the cloth, and are not so susceptible to atmospheric changes. Unless they are removed, the cloth is liable to pucker from one day to the next, particularly if there is a noticeable change in the amount of moisture in the air. It is often impossible to restretch the cloth flat until the selvage is removed, and at times it is difficult to do so at all; hence it is important to remove the selvage before the cloth is first stretched. Since the threads are parallel to the edges, the selvage may be torn off without difficulty, provided reasonable care is exercised to prevent the cloth from tearing at right angles to the desired direction. The width of the strip to be removed is usually apparent, and varies from $\frac{1}{8}$ to $\frac{1}{4}$ of an inch with the different makes of cloth. Enough should be re-

moved to eliminate all the puckers, and to leave the cloth perfectly flat.

1. The cloth should be **tightly stretched** over the drawing to be traced, and held in position by thumbtacks. At least four tacks should be used, one in each corner, two diagonally opposite tacks being placed first. Additional tacks may be used, if necessary, to keep the cloth taut. The tracing cloth should be enough larger than the finished drawing to permit tacking the cloth to the board in such a manner that all thumbtack holes will be cut off when the completed tracing is trimmed to the proper size. The corners of the cloth may be folded under so that each tack passes through two thicknesses of cloth and is thus less liable to tear out. Before the cloth is tacked completely, great care should be taken to see that the paper drawing is so placed that all horizontal lines are truly horizontal in order that the T-square may be used to the best advantage. If the drawing paper is too small to be held by the same tacks, it should be fastened before the cloth is put on. Two tacks are usually sufficient for this purpose and if put at the upper corners will seldom be in the way of the T-square. It may be desirable to use small upholsterers' tacks, driven with a hammer, to fasten the drawing paper to the board, for they will not interfere with the instruments during the making of the drawing or of the tracing. Some draftsmen prefer to use these tacks for holding the tracing cloth in place also, particularly when the T-square or the triangles must be used near the corners; but if the cloth needs to be restretched very frequently the use of such tacks is hardly practical.

2. Before beginning to trace, the draftsman should make sure that **the surface of the cloth** is in condition to receive the ink properly. This may be ascertained by trial on a small piece of the same cloth, upon the margin of the sheet which will be subsequently trimmed off, or even upon one of the lines of the drawing itself. Some of the better grades of tracing cloth will often "take" ink without treatment, and it is preferable to use it that way. More frequently the surface of the cloth is slightly oily and the lines appear ragged. The common method of overcoming this disadvantage is to sprinkle upon the cloth powdered chalk or pumice stone, specially prepared tracing cloth powder, or even talcum powder, and to rub it in with a clean cloth. After a thorough rubbing has spread the powder over the whole surface, another clean cloth, or a brush, should be used to remove completely all the excess powder to prevent the clogging of the pen. If too much powder remains, much of the ink falls upon it rather than upon the cloth, and the lines easily wear away. Furthermore, when too much powder is used, the eraser soon becomes clogged and is made less effective. Accordingly, it is often preferable to "surface" the cloth by means of a sponge eraser, rubbing the grease off instead of covering it or absorbing it with the powder. This is particularly true when the effects of the oil are not very apparent, for the cloth is thus rendered much more satisfactory to work upon. The surface of the cloth should be kept clean, and the path of the pen cleared of all lint, dust, and pieces of eraser. A brush should be constantly available for this purpose, but care should be taken that all ink on the drawing is dry before the brush is used.

3. The draftsman's equipment should include **a good ruling pen,*** and he should not only be familiar with its use, but also be able to keep it in good working order. When not in use the pen should be left with the nibs separated in order to relieve the springs.

The ruling pen should be held between the thumb and forefinger, resting against the middle finger to hold it firmly. The adjusting screw should be held away from the draftsman so that it may be readily turned with the middle finger to change the setting. The nibs should be parallel to the straight-edge, and the handle should be slightly inclined, with the top in advance as drawn from left to right. The handle should remain in a plane through the line to be drawn, the plane being nearly perpendicular to the plane of the drawing. See page 58: 6. The pen should never be pushed backward, even for a short distance, but should always be literally "drawn."

It is important to keep the pen clean. The pen should not be filled until the draftsman is ready to use it, and it should be used practically continuously while it contains ink. Even when interrupted from work the draftsman should take time to wipe his pen before laying it down. This can be quickly done if a large cloth is kept hanging near the left-hand corner of the board or in some other convenient place.

Old tracing cloth, thoroughly washed with soap and water, makes an ideal pen wiper. The wipers which are furnished with bottles of ink are too small to be serviceable. A pen wiper should be free from lint.

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* For more complete treatises on drawing instruments and their use see Blessing and Darling's "Elements of Drawing," John Wiley and Sons, Inc., New York; French's "Engineering Drawing," McGraw-Hill Book Co., Inc., New York; or Kirby's "The Fundamentals of Mechanical Drawing," John Wiley and Sons, Inc., New York.

If ink is permitted to dry in the pen it should be removed before the pen is used again. If allowed to remain it will cause the pen to corrode; this will not only make it more difficult to keep clean, but will eventually prevent precise work. Dried ink should never be removed from a pen with a knife or scratcher for the inner surfaces will become so roughened that the ink will not feed properly. Furthermore, a much simpler and more effective method is to dip the pen in red ink, which will dissolve the caked ink so that it may be wiped off. To avoid all of this, the draftsman should form the habit of always wiping his pen before laying it aside even momentarily, while the ink is still liquid; it takes only a moment for the ink to dry enough to clog the pen, and after this happens it is usually a waste of time to attempt to use the pen again without refilling it.

The ink should flow as soon as the pen touches the cloth. In case the pen has been left unused only for a moment and the ink has dried slightly in the extreme point, the flow may often be started without refilling the pen by making a few short strokes on a piece of paper, wood, or cloth. It is a good plan to wipe occasionally the side of the pen which bears against the straight-edge, for this not only keeps the ink flowing well, but prevents, in large measure, the ink from running under the straight-edge. If, when the pen is adjusted for fine lines the ink cannot be started by the expedients just mentioned, the nibs may be separated temporarily until a heavy line can be drawn, and then readjusted to give the desired width.

The ruling pen is usually filled by means of the quill in the cork of the ink bottle. When used immediately after a lettering pen or other pen in which considerable ink is left, the ink may be transferred from one pen to the other; this saves ink and often time, and prolongs the life of the pen wiper. Similarly, ink may be restored to the bottle by touching the pen to the quill. Care should be taken to avoid getting any ink on any part of the pen other than *between* the nibs, particularly on the part that bears against the straight-edge. The pen while being filled should never be held over a drawing.

It is important not to get too much ink in the pen, particularly for fine-line work. It is difficult to retain a constant width of line if the amount of ink in the pen is greatly increased; it is better to increase the amount slightly before the pen is entirely empty. Experience will show the proper amount to use in pens of different shape under different conditions. For short fine lines the depth of ink above the points should seldom exceed $\frac{1}{16}$ or $\frac{1}{8}$ of an inch.

1. If a pen is not working well, a draftsman should be able to fix the points so that a clear-cut even line of any width from the finest to the coarsest can be drawn. If the points are too dull it is impossible to draw fine lines satisfactorily, and if the points are of uneven length one edge or the other of a coarse line will be ragged. If a **pen is in good condition, it** should be possible to draw lines at different speeds without having them vary in width, or to stop the pen completely and start it again without leaving a pool or other evidence of having stopped it.

To sharpen a pen, rub the outside surface first of one nib and then of the other on a fine oil stone. In order to keep the outside surface of a nib curved, the pen should be moved in the form of a figure eight, with a slight rocking motion, so that the whole edge of the nib is sharpened uniformly. Care should be taken to avoid making flat spots, or sharpening one nib more than the other. After both are sharpened so that the edges show no shiny worn places, make the two nibs of the proper relative length by drawing the pen lightly along the stone, holding it in the same plane as when drawing a line, i.e., approximately normal to the stone, but swinging the handle in this plane to give a curved edge. Test the lengths of the nibs by drawing several heavy lines of different widths. If a line appears ragged on one side, the nibs do not bear evenly, or else one nib is too dull. If the pen will draw heavy lines well, test it for fine lines. The nibs may have been dulled slightly in the process of evening their lengths; if so they should be re-sharpened. If reasonable precautions are taken to avoid excessive grinding the draftsman should be able to obtain edges at the first trial which are even and yet of the right sharpness. The pen should not be left too sharp, for it will either cut the cloth or else make such a deep impression that it is difficult to erase a line when occasion arises.

2. The legs of **the compasses** should be bent at the knuckle joints until the pen and the arm which carries the pivot point are both perpendicular to the plane of the cloth. The compasses should be set to the proper radius by placing the pivot point at the center of the arc and moving the pen until it is immediately above the line to be inked, close to the drawing but not in contact with it. In drawing a curve the compasses should be inclined so that the top of the pen is slightly in advance of the point. A curve should be drawn with a continuous motion, and a complete circle should be closed by carrying the pen a little past the beginning. The weight of the pen is sufficient to insure a good line without additional pressure, and care should be taken to avoid pressure sufficient to alter the radius or to move the pivot so as to cause a crude junction.

3. **The lettering pen** should be well adapted to the individual who uses it, with a pen-holder of suitable size so that the hand will not become cramped. Some draftsmen obtain excellent results with fine points, while others cannot use them without spreading the nibs so as to make lines of uneven width. A long fine stub pen will usually give excellent results for most letters and figures of a structural drawing (page 47 : 5), a fine pen being provided for drawing arrows and arrow heads or special work for which the stub is too coarse. A ball-pointed pen is often used for titles and other prominent lettering. Some draftsmen use a ruling

pen for a lettering, but when this is done a special pen should be kept for the purpose for it cannot be kept in condition for ruling.

1. Two bottles of best quality India waterproof **black ink** should be used, one for instrumental work and the other for lettering. The former should have a quill for filling the ruling pen, but the latter may have the quill cut off, because it is quicker and more satisfactory to dip the lettering pen into the bottle than to use the quill. The bottle with the quill should be kept corked except during the actual filling of the pen, but the other may be left uncorked as long as the lettering pen is in constant use. This would not be feasible if only one bottle of ink were used; so much dust collects in the open bottle, and so much ink evaporates leaving a deposit, that the ink is soon unfit for use in drawing pens, though it may still be used in lettering pens. Each new bottle of ink should be reserved for instrumental work, the remainder of the ink in the former bottles being combined for lettering. Ink should never be diluted with water; if it becomes too thick for use it should be thrown out and replaced.

2. If **red ink** is used, a quality should be selected which can be erased. No red ink can be erased so easily as black ink, but some red inks cannot be removed at all. It should be waterproof; otherwise it is liable to spread when stored in cool vaults.

3. An ink bottle **should never be shaken,** for no benefit is derived and the sediment is stirred up so that it is liable to get into the pen. Furthermore, bubbles are formed in the neck of the bottle which draw the ink from the quill, so that it is difficult to obtain enough to fill the pen.

4. **Frozen ink** is useless and it is usually unsatisfactory when thawed. The bottles should be kept away from windows in extremely cold weather to prevent the ink from freezing.

5. **The straight-edge** should be placed between the draftsman and the line to be inked, so that the near side of the pen bears upon the far side of the straight-edge. Care should be taken to keep the pen against the straight-edge, but to exert no more pressure than necessary to insure this. The pressure should be constant, for otherwise the width of a line will be reduced as the nibs of the pen are pressed together. The pressure of the pen upon the cloth or paper depends upon the sharpness of the pen and the quality of the surface, but it should never be greater than necessary to insure an even line.

6. The draftsman should not attempt to draw **too close to the straight-edge,** lest the ink run under and blot. This distance depends somewhat upon the shape of the pen and the thickness of the straight-edge, but after a little practice the draftsman will learn how close he can work to the best advantage. One-fiftieth of an inch may be taken as a guide to the beginner; this corresponds very closely to the smallest division ($\frac{1}{4}''$) on the scale of $1'' = 1'$. After inking a line one should never attempt to pick up the straight-edge until it has been moved a safe distance away from the line, i.e., toward the draftsman. Otherwise it is difficult to pick it up without letting it slip into the wet ink and cause a serious blot.

7. **An easy posture** should always be assumed before a line is drawn, for it is difficult to do good work in a cramped position. The drawing board may sometimes be turned to advantage. Lines should be drawn with a full arm motion, with the third and fourth fingers resting lightly upon the straight-edge as a guide to give better control. The elbow should not be rested upon the drawing. Near the end of the line the guiding fingers should be stopped just before the fingers which hold the pen, to facilitate stopping the pen at the exact point. This may be done in such a manner that the motion of the pen will not be interrupted. Very short lines may be drawn with this finger motion alone.

8. Each full line should be drawn with a **continuous stroke.** It is important to have sufficient ink in the pen to complete the line. If it is discovered that the ink will give out before the end of the line is reached, it is best to stop abruptly, preferably at some intersection, and to begin at the same point after refilling the pen. If the ink runs out before the draftsman is aware of its being low, the ragged part of the line should be retraced after the pen is refilled. In this event it is important to try the pen on a separate sheet to make sure that the line is of the proper width before applying it to the drawing, for a pen is likely to make a wider line after being cleaned and refilled than when nearly empty.

9. It is well to draw all lines which are of the same width **at one setting** of the pen, if possible, in order to gain uniformity. Even the pens which are made so that they may be opened and cleaned without changing the setting do not always make lines of the same width before and after cleaning. In order to produce a more constant flow, the pen should be refilled before it is entirely empty.

1. Care should be taken to **stop the pen** at the exact end of each line in order to give a finished appearance to the drawing. The pen should be lifted immediately, when the end is reached, to prevent the ink from running out and forming a pool, which it is liable to do, particularly when the pen and the cloth are not both in perfect condition. The pen should always be lifted vertically to avoid a false mark.

2. Lines should be drawn **away from intersections,** as far as possible, rather than toward them, particularly when several lines meet in a common point. A line should never be drawn to meet another line until the latter is perfectly dry.

3. **Heavy lines** which represent web sections (page 37: 2) and other lines which are wider than the main lines or the border lines of a drawing should each be made of two or more component parts, i.e., part of the width should be drawn and *allowed to dry,* then another part, and so on until the full width is completed. If the full width is drawn with one stroke or setting of the pen, the ink will flow so freely that it will take too long to dry, will pucker the cloth, and will make it impossible to get clean intersections because the ink will form a pool where two lines meet. If three lines are used for building up a heavy line, the first two are drawn to form the boundaries of the required line, and they should be so drawn that they are parallel and entirely within the desired width. After these two component parts are dry, the third line may be drawn to fill in the space between them. It should not be necessary to use more than three lines and usually two will suffice. Border lines are generally not so wide that they cannot be drawn with a single stroke, but often two strokes will give better results.

4. In the inking of several **parallel lines,** the triangles or T-square should be used in the same manner as in the penciling of the lines to insure their being parallel. If only a single triangle is used in the attempt to ink over the pencil lines, a slight variation usually results, which is quite apparent. For methods of drawing many parallel lines equidistant, as in cross section lining, see page 37: 2.

5. **All curves** should be inked before the straight lines which are tangent to them. A straight line is tangent to a curved line when the center of the one is tangent to the center of the other, the width at the point of tangency being no greater than the width of either line at any other point. If the straight line is narrower than the curved line, as for example the projection line for a dimension to the extreme outside of a curved surface, the outer edges should be tangent instead of the centers.

6. At practically no time should it be necessary to **wait for ink to dry.** It does not require much ingenuity to find something to do on one part of the drawing while the ink is drying on another part. The man who idly fans the ink with a triangle not only wastes valuable time, but attracts the attention of others to the fact that he is not at work.

7. **On rush work** of revision, or other work which is confined to a small part of the drawing, it may be desirable to ink in the vicinity of wet lines. This may be done by placing a triangle on each side of the wet lines, and then laying across these two triangles a third triangle to be used as a straight-edge. This cannot blur the wet lines since the straight-edge is elevated above the surface of the cloth in such a way that it cannot touch the lines. For a small area it may be sufficient to lay one triangle across the central opening of a large triangle. Care should be taken not to draw a line which will intersect a wet line or figure.

8. **A blotter** should never be laid upon wet drawing ink to hasten drying. In fact the blotter should never be used on a drawing except to absorb superfluous ink from a blot or from a line to be erased, and then only by touching its corner to the crest of the pool, without touching the cloth. If the blotter touches the cloth when wet, it makes the ink penetrate so deeply that it is difficult to erase it, in fact more difficult than if it were allowed to dry without the use of a blotter.

9. In order to obtain the best results in inking or tracing, a **systematic method of procedure** should be followed. The tracing should not be started until the penciled drawing is complete, especially if the tracing is made by a person other than the one who makes the drawing. In case the drawing and the tracing are done by the same draftsman, he should either make the penciled drawing complete, or else work directly upon the tracing cloth, as explained in Chapter XII, page 65. The only modification of this rule is noted on page 66: 1. The order of procedure given in the following paragraphs is recommended.

10. **All the lines** of a drawing should be inked before any of the figures or notes. For widths of lines, see page 37: 1. Ordinarily it is best to ink all the lines which are of one width before changing the setting of the

pen for another width (page 58:9); but if the work is to be interrupted for a considerable length of time so that atmospheric changes might cause the cloth to expand or contract it may seem better to confine the inking to one view, or to so much of the drawing as can be completed before the interruption, especially if the drawing is complicated and the number of intersecting lines is large.

1. All fine line curves should be inked first, and then the rest of **the fine lines,** including the dimension lines, and the center lines, but not including cross section lines, or fine lines which are tangent to heavy line curves. The horizontal lines should be drawn first, beginning at the top and working down the sheet to save waiting for the ink to dry; next the vertical lines, beginning at the left and working toward the right; and then all other lines of the same width, working in some systematic order to prevent the omission of any.

2. **The heavier main lines** of the drawing should be inked next, following the same order as given for the lighter lines in the preceding paragraph.

3. **The rivets and holes** may be put in next. These should never be drawn until the lines are drawn, for it is simpler and more satisfactory to center the rivets on a line than it is to draw a line through the centers of a row of rivets. The rivets should always be drawn approximately to scale with a bow-pen or a riveter.

Few bow-pens can be adjusted to make circles small enough for the general requirements of structural drafting, and riveters are much better adapted to the purpose. The riveters fitted for ink only, i.e., riveting pens, are recommended instead of those which are interchangeable for ink and for pencil. A fixed needle point is held vertically while the revolving pen is twirled around it. The pen can be set to make a circle which is so small that it is virtually a period, and can be lifted out of the way while the needle point is being centered. Such a riveter should form part of the equipment of every structural draftsman.

The appearance of any drawing is marred by freehand rivets and holes. For the conventional method of indicating rivets and holes, and for the sizes of the circles, see page 40:6. All shop rivets of the same diameter should be drawn with one setting of the riveting pen in order to make them of uniform size, and the same precaution should be observed in drawing the holes for the field rivets. The latter may be filled in solid at once, but considerable time is required for the little puddles to dry so that they will not smear, and for this reason it may save delay to wait until the rest of the drawing is completed before filling in the holes or else to fill them in a few at a time while dimensioning. Some draftsmen prefer to draw all **arrow heads** before putting any of the dimension figures on the drawing; in this case the open holes may be filled in at the same time. See page 47 : 4 for the styles of arrow heads. When filling the circles for the field rivets it is an excellent plan to slip a blotter under the cloth to absorb any ink that may run through the holes made by the needle point of the bow-pen or the riveter. By using a little forethought the draftsman can usually plan his work so that the ink will be dry on part of the drawing by the time that he completes the holes and arrow heads on the rest, so that he can proceed at once with the dimensions.

4. After the arrow heads have been inked, **the dimension figures** may be put on, but this should be done systematically in order that none will be omitted. On some drawings it may be well to ink them in the order of importance, and on others in the order in which the shopmen will use them, thus making sure that sufficient dimensions are given. If the penciled drawing is well made, however, the dimensions on one view may be inked first, one line of dimensions at a time, and then the dimensions on the next view, and so on. When a detail is dimensioned chiefly in one view, but has a gage or a few similar figures in another view, it is well to ink these figures in connection with the rest of the detail, rather than to wait until the remainder of the view is inked.

5. After the drawing has been dimensioned, all **material should be billed** as outlined in Chapter VII, page 43; then the **list of members** required and all **notes** should be inked, first the specific notes on various parts of the drawing, then the general notes, as outlined in Chapter IX, page 52. For all of this work pencil guide lines should be used, not merely by beginners, as students are wont to believe, but by experienced draftsmen as well. Guide lines are essential to all good lettering; like the carpenter's staging, they are used by the best men as well as by the novices, but, like the carpenter's staging, they should be removed after the work has been completed. To obviate the necessity of ruling and erasing lines on each drawing, a small sheet ruled once for all with parallel lines at proper distances apart, may be slipped under the tracing cloth where the lettering is to be, and used instead of the pencil guide lines. It

is convenient to have a large number of parallel lines on this sheet not only to provide for notes which have many lines, but also to simplify the placing of the sheet in the desired position. Diagonal red lines drawn at the standard slope for inclined letters are of great assistance in maintaining a uniform slant in lettering. A convenient size for this sheet is $5'' \times 8''$; if smaller it is difficult to place it under the cloth in the right position. Sheets with different spacing may be made for various sizes of letters, or different spaces may be combined on one sheet, if the change from one to the other is made conspicuous. Uniform spaces of $\frac{1}{16}''$ make good units, for a single space may be used for small notes, two spaces for Required Lists, and three spaces for titles. Simple freehand letters should be used entirely, and the draftsman should practice lettering until proficient, for a good looking drawing may be easily marred by crude lettering.* Slanting letters are preferable to vertical letters because deviations from a uniform slope are less apparent than deviations from the vertical. Most draftsmen can letter more rapidly with sloping letters than with vertical letters.

1. Finally **the title** should be traced and **the border inked. For suggestions** for making the title and the border see page 54 : 1–7.

2. As soon as **the tracing** is removed from the board it **should be inverted** so that the draftsman may ascertain if any ink has passed through defects in the cloth, or through holes made by the instruments. All such blots should be removed, for they would cause spots on the prints and thereby mar the appearance just as if on the face of the tracing; they might cause serious trouble if an important figure was thus altered or obliterated.

* The beginner should obtain some standard book on lettering, such as Reinhardt's "Lettering for Draftsmen, Engineers and Students," D. Van Nostrand Co., New York; Blessing and Darling's "Elements of Drawing," John Wiley and Sons, Inc., New York, or French's "Engineering Drawing," McGraw-Hill Book Co., Inc., New York.

CHAPTER XI

ERASING

SYNOPSIS: It frequently happens that parts of structural drawings must be removed on account of mistakes or changes. Every draftsman should learn to erase in such a manner as to leave the least possible evidence of erasure, even when several erasures are made in the same place on a drawing.

1. **To erase** any part of a drawing **properly** is just as important as to ink or trace properly, and the draftsman should expect to devote a considerable part of his time to painstaking and careful erasing. In the first place, he must correct his own mistakes and errors of judgment, and in the second place, he must frequently make changes which may be due either to mistakes of others or to changes in design. Many good looking drawings are practically ruined by careless erasing.

2. **The object of erasing** is not merely to remove part of a drawing, but to remove it in such a manner that other lines and figures may be placed in the same spot without having the change apparent on the blueprint. This can be accomplished only by a person who fully realizes the difficulty, and who exercises great care and patience.

3. **Erase Willingly.** — The first and often the only indication of friction between a beginner and his superiors usually arises because he objects to erasing. While he should never make changes until he approves them, yet he should stand in readiness to believe that the more experienced checker has good reasons for most changes and he should try to see his point of view; moreover, the objections of the novice are more likely to be heeded, if, instead of attempting to convince the checker that minor changes should not be made because they involve too much erasing, he reserves his arguments for more important matters. The friction should be between the draftsman and the drawing *not* between the draftsman and the checker. To argue over erasing is usually futile, and more time is lost than would be used in making the corrections at once; furthermore, the draftsman is liable to lower himself in the estimation of the checker. He should be more tactful.

4. The draftsman should profit by the criticisms of the checker, and **guard against mistakes** similar to those which have been corrected on former drawings. Erasing mistakes will often help him to remember them. A new man is judged not so much by the mistakes he makes, as by the mistakes he makes a second time; if he is careful not to repeat any mistake it will not be long before he will outrank the men who are not so careful.

5. Ink should be removed from tracing cloth by means of an eraser. **The secret of erasing** on cloth is to rub so lightly and slowly and with such frequent rests, that the cloth will not become noticeably heated; if it becomes a bit warm the preparation which makes the cloth transparent will become softened so that the eraser will remove it. If this preparation is once removed the cloth is made opaque so that a white spot will show on the blue print, and the surface cannot be restored for further inking. A little experience will show how many strokes may be made without heating the cloth, the temperature of which may be tested with a *dry* finger. When there are several different parts to erase, the draftsman can rub a few strokes in one place, then in another, and so on, until enough time has elapsed to allow the first part to become entirely cool, when the process may be repeated. The cloth should be supported by a smooth hard surface before the eraser is applied; unless the drawing board is unusually free from holes and dents, a triangle or something similar should be placed under the part to be erased.

1. **The Eraser.** — Either an "ink eraser" or a "pencil eraser" may be used for removing ink from tracing cloth. The former is more effective but it is liable to scratch or injure the cloth. One may obtain more satisfactory results with a pencil eraser if one has abundant patience and avoids excessive speed so that the cloth never becomes heated. When the ink is quite thick part of it may be removed with an ink eraser and the remainder with a pencil eraser. Whenever an ink eraser is used it should be followed by the use of a pencil eraser to clean the drawing properly. The center of contact should be kept on the ink to be removed lest the adjacent cloth become seriously damaged before the fact is realized. An eraser will often become soiled or clogged from use, particularly when used on heavy lines or on smooth paper. It should be cleaned by being rubbed on clean rough paper or on a clean portion of the drawing board reserved for the purpose. The white or "ruby" erasers are firmer and better adapted to the removal of ink than the "emerald" ones; they are also more likely to be self-cleaning.

2. A special **ink eradicator** for tracing cloth is on the market but it should not be used too generally. It should not be used for small erasures, since it cannot be confined to small area to good advantage and it should never be used if any erasures or scratches have been made previously within the same area. For taking out a whole detail or other large portion of a drawing the liquid eradicator usually proves satisfactory.

3. **A knife or metal scratcher should not be used** on a drawing in place of an eraser. The surface of the cloth is so damaged that it is impossible to re-ink properly the portion which has been scratched. The cloth often becomes so opaque where a knife has been applied that the scratched portion shows on the print almost as distinctly as if the ink had not been removed, and furthermore, the lines appear ragged. A very sharp scratcher in the hands of an expert can be used sparingly to advantage; but as a rule, the use of a knife or scratcher is a confession of laziness, for there is nothing to recommend it except that it is sometimes easier to scratch out a small section of a line than it is to erase it and then have to replace the surrounding lines which may be erased also. This advantage is offset by the fact that drawings frequently have to be retraced when a later revision necessitates inking where a knife has been used. Erasing with a knife nearly always involves the risk of injury to the cloth, and is in this sense a dangerous habit which is not justified by the results; better results are almost invariably obtained when an eraser is used.

4. When erasing a figure, a rivet, or small detail, which is close to other lines or figures, one may use **an erasing shield** to protect the surrounding parts, and thus simplify the filling in afterwards.

A shield may be made by cutting any desired size and shape of aperture through a thin, tough card. Erasing shields of nickle-plated brass or of steel may be bought, those of steel being recommended, if used enough to keep them from rusting, because they are much more durable.

5. **A brush** should be used to remove all pieces of eraser and other foreign matter from the drawing as soon as the desired parts have been erased. If pieces of eraser adhere to the cloth they may be brushed off more easily if a little tracing cloth powder (page 56 : 2) is sprinkled over them.

6. **The surface of the cloth** where any erasure has been made must be treated before ink is applied, or the ink will spread. Although chalk or pumice stone are frequently used, it is better to polish the cloth, for the new lines will be more durable if on the cloth itself, than if partly on the powder. A triangle or other hard surface should be slipped under the cloth, and a smooth, clean piece of soapstone or celluloid rubbed over the erased area until the cloth shines. Other hard surfaces may be substituted for the soapstone, but they are likely to soil the cloth. An end of a celluloid triangle serves very well if the corners are rounded and used for this purpose only, care being taken to avoid the worn part when the triangle is used as a straight-edge.

7. After erasures have been made and the cloth has been polished, all **lines and figures** which have been **erased by mistake** should be replaced whether there is anything to be added or not. This point is frequently overlooked, especially if only a small portion of a line is erased, but it requires only a few such omissions to mar the appearance of a drawing. In order to prevent blurring or blotting, especially if the erased surface is not very smooth, the heavier lines should each be built up by making several fine lines until the desired width is obtained, no line being drawn until the preceding one is dry.

1. If **pencil lines** are used on the tracing cloth, they may be left until the tracing is finished, and then removed along with any accumulated dirt. The author prefers a soft sponge eraser for this purpose, particularly if it was used at the beginning instead of powder to surface the cloth (page 56 : 2). Care should be taken to rub *between* the lines as far as possible, rather than across them; rubbing the lighter lines tends to make them too dim to print well especially if the cloth has been surfaced with powder. The pencil lines may also be removed by rubbing the surface of the drawing with a cloth dampened with benzine; the benzine does not affect the waterproof ink. Care should be taken to use only clean cloths, lest the whole drawing be made dingy. Many draftsmen prefer the use of benzine on account of its simplicity; but for various reasons many companies do not furnish it. The author feels that benzine renders the cloth more liable to crack in handling and thus the usefulness of a drawing is somewhat impaired.

CHAPTER XII

DRAWING DIRECTLY IN INK ON TRACING CLOTH

SYNOPSIS: The advantages of this method over the method of drawing on paper and then tracing are shown; suggestions are given regarding the use of this method.

1. **Method Recommended.** — Drawings may be made on paper and then inked or traced, or they may be made directly on tracing cloth. Some companies adopt one system, some another, while still other companies allow each draftsman to choose for himself. The majority of structural drawings are so similar to drawings already made that it is possible to draw many lines in ink without drawing them in pencil first. For this reason the method of making drawings directly on tracing cloth is recommended whenever practicable.

2. Some of the companies that adopt the system of penciled drawings employ **tracers,** who simply trace the drawings made by other men. The tracers are apprentices, recent college graduates, or others of limited experience who are thus enabled to learn the points peculiar to the structural company for which they work as well as the usual conventions of the drafting room. Soon after a tracer becomes proficient he is usually allowed to make drawings himself, and his place is taken by a new tracer. By this process it is difficult to keep the tracing up to the proper standard and the tracings may be issued in very poor form; the detailer may have to spend considerable time in preparing the tracings for the checker, or the checker may have so many errors to indicate that he cannot work efficiently. It is poor economy to allow good men to spend valuable time on drawings, only to have much of the meaning and good appearance lost through careless tracing; hence it is often better for draftsmen to make their own tracings. If the draftsman were to trace his own pencil drawing he would not need to make it quite so complete as if another person were to trace it; but even less penciling would be required were he to make his drawing directly on the cloth instead of on paper. In this way he might ink many lines and figures without first making them in pencil.

3. **Arguments.** — Although it is more serious to make false lines in ink than in pencil, this should not prevent any careful draftsman from adopting this method of inking much of the drawing without previously penciling it. He should cultivate accuracy by strict attention to his work, and he should check his work repeatedly to avoid carrying a mistake so far that it will entail extensive alteration. When first attempting this method, the draftsman should ink only the lines and the figures which he is sure are right; on subsequent drawings he will find that a larger number may be inked, and it is believed he will gradually become an enthusiastic advocate of this method. Most men who are opposed to the method have not given it a fair trial; or they have attempted to ink too much, and naturally no time has been saved. The men who still persist in making complete pencil drawings after years of experience are being outranked, for the most part, by men of equal ability who work more efficiently by drawing directly in ink on the cloth. Even if an occasional drawing has to be retraced on account of mistakes made in inking directly on cloth, this fact should not be given too much weight, for the chances are that the same mistakes would have been made in pencil. If that part of the pencil drawing which is correct can be preserved and traced, so also can the same part of the inked drawing be retraced just as quickly. Doubtless a little more time is taken in making the original drawing in ink than would be taken to make it in pencil, but

since comparatively few drawings need be retraced, this loss is more than balanced by the time saved in making other drawings directly in ink on the cloth; this gain is approximately the difference between the time required for drawing once in pencil and then tracing, and the time required for simply drawing in ink once.

1. **Not all drawings are well adapted to this method** of drawing directly in ink. If a large number of pencil lines must be drawn, many of which may have to be erased, better results would probably be obtained by making a pencil drawing on paper. Sometimes the **two methods may be combined** to advantage. Thus, if a drawing is so complicated that the best positions for the different views must be determined by trial, the preliminary work may be done in pencil on paper; but as soon as the views are finally located and all necessary pencil lines drawn, this much of the drawing may be traced, and the remainder completed on the cloth just as if all the lines were so drawn.

2. **A drawing must be carefully planned** in advance if the whole or any part of it is to be inked without previous penciling, in order to insure a good arrangement and to avoid crowding. This is not an argument in favor of a complete pencil drawing, because such preliminary planning should be done also before a pencil drawing is started, so as to avoid the necessity of shifting the cloth while tracing to effect a change in the arrangement. The number of views and the number of dimension lines should be determined, and also the extreme dimensions of each view, including the main member and all projecting parts. If any sectional views are to be placed in breaks in other views their positions should be anticipated to avoid erasing spaces for them later.

3. A sheet of clean **paper** should be **placed underneath** the tracing cloth before a drawing is started on the cloth; the paper makes the lines more distinctly visible and also covers the thumbtack and other holes in the board so that there is less danger of holes being punched in the cloth by the pencil.

4. **Illustration.** — As soon as the number of views and dimension lines and the main dimensions of a member are determined, points may be plotted to indicate the position of each line that extends practically the full length or depth of the member in each view; pencil lines may be drawn if necessary to show where these long lines stop. Then these full length lines, both dimension lines and lines of the main drawing, may be drawn in ink without being drawn first in pencil, with a corresponding saving of time. In case some of the lines represent parts which are to be behind details to be added later, these lines may be drawn in pencil or omitted altogether until the details have been located; then the lines may be inked, with dashes to represent the invisible portions. For example, let us consider the drawing of a plate girder with cover plates (Fig. 102). Suppose that front, top, bottom, and end views are required and that the girder is symmetrical about the center line. From a preliminary sketch we find how many dimension lines are needed. Points may now be plotted with due regard to margins and spaces between views to insure a good arrangement. Vertically, these points will show the position of (1) all full length dimension lines; (2) the three lines of each flange angle in the web view, with the corresponding rivet lines (usually at the standard gage); (3) the eight lines in the top view for the cover plate and flange angles, with the corresponding rivet lines (usually two in number); (4) same as (3) for the bottom sectional view. Horizontally, these points will show the position of (5) the left end and the center line of the girder; (6) the cover plate and flange angle lines of the end view which are the same as those in the top view, and in addition the lines of the stiffening angles and their rivet lines; (7) the full depth dimension lines. Now the pen may be filled and set for fine lines, and lines may be drawn in the following order: (1) a continuous vertical line at the end of the girder drawn from the bottom view to the top view; (2) a dot and dashed line at the center drawn from the bottom view to the top view; (3) all horizontal dimension lines and rivet lines, including lines from the end view to the front view to indicate the depth from back to back of flange angles; the rivet lines should extend beyond the end of the girder to the proper dimension lines; (4) the rivet lines in the end view and the vertical dimension lines. Now the pen may be set for wider lines, and the following lines may be drawn: (5) the cover plate lines of the top view, provided the plate extends full length; (6) the lines which show the outstanding legs of the flange angles in the web view; (7) the heavy web line and the other lines of the bottom view, except the cover plate lines which cannot be drawn until the lengths are determined; (8) the vertical lines of the end view (except the dashed

lines) and the end line of each of the other three views; (9) the horizontal lines of the end view. The pen may now be set for slightly narrower lines (page 37 : 1) and the dashed lines may be drawn (10) for the flange angles in the top view, and (11) in the end view. The main dimensions may now be recorded in ink. Thus the drawing is well advanced without the use of a single pencil line. As soon as the stiffening angles are located, points may be plotted to show the three lines of each angle and the corresponding rivet lines. The rivet lines may be inked, then the stiffening angles in all three views, and the remaining lines of the flange angles in the web view which must be dashed behind the stiffening angles now located. As soon as the spacing of the rivets in the web view is determined and the totals checked in each panel as well as in the full half-length, the necessary rivets may be plotted and the corresponding lines and dimensions inked. Similarly the other views can be completed, and when the cover plate lengths are definitely determined they may be shown in the top, front, and bottom views. The rivets and holes may be shown, the material may be billed and the notes and title may be made directly in ink without being penciled.

CHAPTER XIII

RIVET SPACING

SYNOPSIS: Rivets must be spaced to conform to general rules and specifications which are in common use; such rules are given in this chapter. The spacing is also dependent upon the number of rivets required under different conditions, as explained in the chapters of Part III.

1. "**Rivet spacing,**" as a general term, refers to the dimensions which locate either shop or field rivets. These dimensions extend invariably to the centers of the rivets. "**Rivet pitch**" is a more specific term usually limited to the spacing which locates the rivets that connect the component parts of a built member in the direction *parallel* to the longitudinal axis. This term is most frequently applied to the flange rivets of plate girders, in which the pitch at different points must be determined from the given loads, as explained in Chapter XXXVII, page 241.

2. **General rules** for the spacing of rivets are given in this chapter to conform to those in common use. The spacing is also necessarily dependent upon the number of rivets required to satisfy the conditions of loading and other considerations which are discussed in different chapters of Part III.

3. Each structural company adopts a **set of standards** for the guidance of its draftsman in order to make the drawings more uniform. Each draftsman should follow, whenever it is feasible, the standards of the company for which he works.

4. **The specifications** which accompany each contract should be carefully read, and the rivet spacing should never violate any clause therein. For the most part the different sets of specifications are quite similar; the rules of this chapter conform to the majority of them.

5. **Standard Gages.**—The flanges of I-beams and channels are so narrow that the usual rules for clearance and edge distance cannot be applied transversely. Standard gages which will best meet all requirements are therefore adopted. The gages given in the tables on pages 298 to 302 inclusive are in common use, although the standards of some companies differ slightly. Standard gages for angles are also adopted as shown on page 303. These standard gages should be used in all places unless there is good cause for deviation. However, it is usually better to change the gage slightly on one drawing than it is to make the distance between rivets on two or more drawings result in sixteenths or eighths, as in connection angles for beams and girders (pages 83 : 6 and 106:3), or in the flanges of girders and columns (pages 106:3 and 136:1). The rivets in diagonal bracing are often placed in the centers of the angles instead of at standard gages (page 139:3). The two rows of rivets in a 6-inch angle are often separated more than usual to accommodate the spacing on the members to which they connect, as in the base angles of columns (Fig. 133) or in struts (Fig. 147).

6. **The minimum spacing** of rivets should be such that the strength of the metal between rivets fully develops the strength of one rivet. The minimum pitches for the rivets in the flanges of plate girders should be determined from the table on page 306 in accordance with the conditions of each problem, as explained on page 255:2. In most other cases it is more convenient and sufficiently accurate to use a certain minimum space for each different diameter of rivet regardless of other conditions. The values most commonly specified for absolute minimums are either "three diameters," i.e., three times the diameter of the rivets, or else

the "usual minimum" tabulated on page 305; these specifications differ only for the smaller rivets. A preferred minimum is also shown to be used in work of the better class. These values are based upon average conditions. (Compare with the values for rivets in a single line given on page 306.) No two rivets should be placed closer together in any direction than the proper minimum space. The minimum spacing for staggered rivets in tension members should be taken from the diagram on page 305, as explained on page 209 : 1.

1. **The maximum spacing** of rivets differs not only with their diameter, but with the type of member and the position of the rivets in the member. When rivets are staggered on two lines, as in the flange angles of girders or columns, the maximum pitches given below refer to the distance from a rivet on one line to the next rivet on the other line measured *parallel* to the rivet line as if the rivets were on a single line. (Compare the next to the last sentence of the preceding paragraph). (*a*) In general, the maximum pitch of rivets measured parallel to the principal axis of a member is 6″ for 1″, ⅞″ or ¾″ rivets, 4½″ for ⅝″ rivets, and 4″ for ½″ rivets. Many specifications limit the pitch of ¾″ rivets to 5″. (*b*) The pitch should not exceed 16 times the thickness of the thinnest exposed plate or other shape. (*c*) For girders, which support moving loads applied to the flanges, as crane girders, or stringers of bridges and viaducts, a maximum pitch of 4 or 4½ inches is usually specified for the flange rivets. (*d*) The pitch of the rivets which fasten the component parts of a compression member together should not be more than four diameters at the ends of the members and opposite the connections of heavy loads. This close spacing should extend the full depth of such connections, and at the ends for a distance which is variously specified as equal to one, one and a half, or two times the width of the member; the mean value of one and a half may be used unless otherwise specified. (*e*) Rivets in tanks should not exceed about four diameters to make the joints watertight.* (*f*) Rivets which do not transmit much axial stress may be spaced farther apart than the values given above; thus the rivets which fasten skew-back angles for floor supports to the webs of beams and girders may be 1′-0″ apart, rivets which connect stiffening angles to channel struts or stiffening channels to crane beams 1′-6″, countersunk rivets in column bases from 9″ to 1′-0″, and stitch rivets as indicated on page 69 : 4.

* See table of spacing for watertight joints in Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

2. **Wide Cover Plates.**—The rivets in the flange plates of a compression member are usually placed in two rows unless the distance between the rivet lines, measured at right angles to the principal axis, exceeds forty times the thickness of the outside plate; in this case four rows would be used. If two or more plates project 3″ or more beyond the edges of the angles, an extra row of rivets must be used to fasten them together, the pitch being twice that of the rivets which connect the plates to the angles.

3. **"Edge distance"** is a term applied to the *perpendicular* distance measured from the center of a rivet or hole to the edge of any structural shape. When possible the edge distance should be at least one and a half diameters, and preferably two diameters, as tabulated on page 305. This is especially important in the direction of the line of stress. Smaller edge distances are unavoidable in the flanges of the smaller beams (page 68 : 5), and it may seem best to use them in comparatively light work in places where the available space is limited, but it is usually best for the novice not to make such exceptions without due counsel. The edge distance should not be more than 5″ nor more than eight times the thickness of the thinnest exposed plate or other shape.

4. **Stitch Rivets.**—A member composed of two angles should have them riveted together at frequent intervals in order that the stress may be distributed equally between the two angles. If the angles are separated on account of connections to gusset plates, a washer is placed between them at each rivet in order to maintain a uniform distance between the angles. These equalizing rivets with or without washers are called "stitch rivets." They need not be dimensioned, but the proper number should be shown or else the approximate spacing should be noted. The distance between the last rivet of one connection and the first rivet of the next connection should be divided approximately equally. Stitch rivets are spaced from 2′-6″ to 3′-0″ apart in tension members, but from 1′-6″ to 2′-0″ apart in compression members on account of the tendency of the latter to buckle. The specifications for railway bridges sometimes limit the spacing of stitch rivets in tension members to 1′-0″.

1. **Lattice bars** are used in light compression members or diagonals instead of web plates or cover plates, and on the under side of chord members where it would be impractical to use cover plates. Tie plates should be used at the ends of each group of bars. For the sizes of tie plates and lattice bars, see page 216 : 2–3. For the method of billing and the method of manufacturing lattice bars see page 45 : 2. For the method of representing and dimensioning lattice bars see pages 40 : 3 and 50 : 6. Either "single latticing" or "double latticing" may be used as shown on page 315, double latticing being used when the distance between rivet lines is more than 1′-3″. A rivet is placed at each intersection of double lattice bars. The inclination of double lattice bars with the longitudinal axis of a member should not be less than 45°, i.e., the distance from center to center of rivets in any bar, measured parallel to this axis, should not exceed the corresponding distance measured at right angles to the axis. The inclination of single lattice bars with the longitudinal axis varies from 60° in important members to 45° in comparatively light and unimportant ones; for most work an inclination of from 50° to 55° proves satisfactory. When single lattice bars are used on opposite sides of the same member the bars should alternate, as shown in Fig. 129. The clearance between the end bar and the tie plate should preferably be from ¼″ to 1½″, or else the bar should overlap the plate with a common rivet. The spacing for different groups of lattice bars on the same or similar members should be made so that the bars are interchangeable as far as possible.

2. **Practical Points.** — (*a*) Every separate piece should contain at least two rivets even if one rivet is strong enough, because a single rivet is not sufficient to hold the piece in position properly. (*b*) Rivets should be spaced with due regard to the appearance of the finished member; for example, a single small pitch between two large pitches in a girder flange is conspicuous (page 70 : 4). (*c*) When multiple punches are to be used, the rivet spacing should be given so that the punches can be used to the best advantage. In multiple beam punches the spacing is usually fixed. In multiple plate punches the spacing may be made to correspond to the drawing, but the work may be facilitated if the holes for intermediate connections are made to line up with the other holes. (*d*) Templets are often made to serve for several different members; this fact should be borne in mind when the rivets are spaced. Long lines of rivet spacing on similar members should be kept alike as far as possible, the different spaces being kept near together, preferably at the ends so that different short templets may be used in conjunction with one long one. The differences may often be made in the same templet by boring different sets of holes; the centers of these holes should not fall less than ½″ apart or they will interfere with each other.

3. **Usual Spaces.** — The rivets which connect the main component parts of a member are spaced as far apart as is compatible with the conditions outlined in the preceding paragraphs, in order to minimize the number of rivets to be driven. But the rivets which connect one member to another, or part of one member to another part, are placed at the usual minimum or the preferred minimum distances as far as possible in order to reduce the size of the connecting material.

4. **Continuous Rivet Spacing.** — The following suggestions may aid the beginner in spacing long lines of rivets which extend virtually the whole length or depth of a member: — (*a*) First locate all rivets which are determined by given conditions, such as those in connections to other members, those near the ends of compression members, or those which must line up with other rivets on account of fixed gages or working lines; then complete the spacing of the intermediate rivets as follows: (*b*) If the number of intermediate rivets is determined by the stress the rivets are spaced approximately equidistantly. The available distance is divided into the proper number of spaces (one more than the number of intermediate rivets); unless the result is a multiple of ¼″, the nearest ¼ is usually chosen for part of the spaces, another value being used for the one or more remaining spaces. Usually one odd space will serve to balance the line satisfactorily unless the spacing should be kept symmetrical as in the stiffening angles of a plate girder when two should be used. (*c*) If the intermediate rivets are spaced at a fixed pitch (usually the maximum allowed) as in columns or chord members, the number of such spaces is determined and any remainder is noted. This remainder may be inserted as a special space provided it is larger than the adjacent space; otherwise it is better to add it to one or more of the maximum spaces and subdivide the sum; this should preferably be arranged so that not more than one space results in sixteenths, and

so that all of the spaces are smaller than the fixed pitch but equal to or larger than the adjacent space for the sake of appearance. When more "balancing spaces" than one are used they may all be placed at one end of the group, or part may be placed at each end. (*d*) If the rivet pitch changes at intervals as in the flanges of plate girders, enough spaces of each pitch are used to extend the proper distance from the end (page 241 : 5). Near the center, enough spaces of the last pitch must be used to complete the total length; in case a remainder is left, one or more odd spaces may be inserted at the last change in pitch, but the odd spaces should be larger than one pitch and smaller than the other for the sake of appearance. The odd spaces should not be placed at the center of the girder because a few small spaces in the middle of a group of large spaces would not look well. The spacing should be made symmetrical about the center line; the center line should therefore fall either at a rivet or midway between two rivets, whichever gives the better arrangement of balancing spaces. If the girder is divided into fixed panels by stiffeners, care should be taken that the spaces in each panel total the proper amount, and that ample driving clearance is allowed for all rivets (page 73 : 5). (*e*) For the benefit of the shopmen sixteenths and eighths should be avoided whenever practicable. For instance, rather than make ten equal spaces in sixteenths, eight or nine spaces can be made alike leaving one or two odd spaces, not more than one of which involves sixteenths. (*f*) After a line of rivet spacing is completed it should always be totaled to make sure that the sum equals the proper amount;* similarly the sum of the spaces which subdivide any other dimension should equal that dimension. (*g*) The multiplication table for rivet spacing on page 307 may be used to advantage in this work.

* There is a small adding machine (The Architects' Calcumeter) on the market, which is admirably adapted to this purpose, but at present its price is beyond the reach of the average draftsman.

CHAPTER XIV

CLEARANCE, AND ERECTION CONSIDERATIONS

Synopsis: There are many points which the draftsman should consider in order to make the erection of a structure not only possible but comparatively easy. Clearance should be allowed so that members or parts of members may be assembled without interference, and so that rivets may be driven by machine.

1. **Clearance** should be provided wherever possible to facilitate the assembling of the component parts of members in the shop, and the erection of the whole members in the field. Clearance is of greater importance in field connections than in shop work because of the difficulty in handling the larger pieces which are involved and which must be put into comparatively inaccessible places. It is much simpler to trim a small piece in the shop than it is to cut a whole member in the field. The more common places where clearance should be provided are (*a*) between the component parts of a member, (*b*) between the projecting parts of a member and the members to which it is to connect, (*c*) at the ends of a member which is to be inserted *between* the faces of the supporting members, and (*d*) between each rivet and any projecting part which might interfere with the use of a riveting machine.

2. **Provision for Overrun.** There is often a variation between the actual size of a structural shape and the size indicated on the corresponding drawing. This variation may be the result of the methods of rolling the material at the mills, especially angles, as explained on page 25 : 1; or it may be due to inaccurate cutting either at the mill or at the shop. In any event, due allowance should be made so that the assembling of the different parts will not be made unnecessarily difficult. For example, the rivets which connect the diagonal members of trusses, latticed girders, or bracing systems to the gusset plates should be so placed that these members may be cut short enough to avoid any interference with the chords or other members. This provides for any variation in the lengths of the diagonal members or in the depths or widths of the chord members. See pages 76 : 1 and 138 : 5. Similarly I-beams and channels are ordered short enough to allow for overrun as explained on page 88 : 1, to avoid the necessity of recutting them. If the members are assembled in the shop the usual clearance is $\frac{1}{4}''$, but if assembled in the field the clearance is increased to $\frac{1}{2}''$. In order to eliminate sixteenths and preferably eighths from the billed lengths of the members the above values may be increased by $\frac{1}{16}''$ or $\frac{1}{8}''$, or in some cases reduced by $\frac{1}{16}''$.

3. In some places **tight fits** are necessary, but this work involves additional expense and should be avoided whenever possible. Stiffening angles of plate girders are usually fitted so that the outstanding legs are in contact with the flange angles. These angles must not only be cut carefully to length, but they must be cut to clear the curved fillets of the flange angles (page 26 : 1). Similarly, stiffeners are used under seat angles and in similar places.

4. **Projecting parts** should be arranged so that they will not interfere with the erection of a member. Clearance should be left between members which connect independently to the same member. One or more angles of a member may have to be cut shorter than the remaining angles, as for example the chords of *LG* 2, Fig. 111. Similarly a member may have to be notched or blocked out as in Figs. 87 and 104. The amount of clearance should be at least $\frac{1}{2}''$.

1. **Erection Clearance.**—When one member is to frame at right angles between the faces of two other members and is to be connected by means of angles riveted to the webs, as for example *B* 10 (Fig. 87) and *S* 1 (Fig. 98), the extreme distance from back to back of connection angles should be made less than the clear distance between the faces of the supporting members in order to allow "erection clearance." It is not always possible to allow sufficient clearance to permit the member to be swung into position without moving the supporting members; for if the ratio of the width of a member to its length is large, the necessary clearance would be so great that the surfaces could not be drawn together to make a satisfactory riveted joint. But even if the supporting members must be spread while the other member is being inserted, a little clearance is desirable as a safeguard against an overrun which would prevent restoring the supporting members to their proper positions. The amount allowed varies from $\frac{1}{16}''$ to $\frac{3}{16}''$ at *each* end; some companies contend that $\frac{1}{16}''$ is practically negligible on account of the paint, scale, and careless shopwork which may counterbalance it; but from a larger clearance may result loose joints, or else the columns and girders which support long lines of such beams may be drawn out of plumb. For most work $\frac{1}{16}''$ or $\frac{3}{32}''$ at *each* end should be allowed for erection clearance; thus: in order to make the length back to back of angles free from sixteenths, a *total* clearance of $\frac{3}{16}''$ should be subtracted if the *clear* distance between supports results in sixteenths; otherwise $\frac{1}{8}''$ should be subtracted. No erection clearance is required when one end of the member is wall bearing or otherwise free to move, or when the member frames diagonally between two other members.

2. **Seat Angles.**—The end of a beam or a girder should be supported entirely by end connection angles, or else by a seat angle or some form of bracket, but never by a combination of the two because of the difficulty in making both act at the same time. Erection seats should be provided to support girders with end angles until the rivets are driven (*F*1, Fig. 99); erection seats may also be provided for heavy beams or for beams which are to be connected to opposite sides of a web plate by the same rivets. Usually an erection seat is shown $\frac{1}{8}''$ lower than the bottom of the girder so that if inaccurately set it will not prevent the placing of the rivets. When seat angles or brackets carry the whole load, top angles or side supports should be provided to prevent the beams or the girders from overturning. Top angles should be shipped bolted so that they may be removed during erection if desired. A clearance of $\frac{1}{4}''$ should be left between the tops of the beams and these angles to provide for any increase in the depth of the beam on account of the spreading of the flanges or the use of worn rolls during manufacture; a similar clearance of $\frac{1}{4}''$ should be left between the angles and the tops of girders.

3. **Holes for anchor bolts** for columns and girders should be made $\frac{5}{16}''$ or $\frac{3}{8}''$ larger than the bolts. This either simplifies placing members on bolts which are already set, or else provides for drilling holes in the masonry after the steel work is in position. The holes should be located with these points in view.

4. **Other Connections.***—The extreme width or depth of web members, top struts and laterals, and other members which must be inserted between two gusset plates, should be made $\frac{1}{8}''$ less than the clear distance between the plates, thus allowing a clearance of $\frac{1}{16}''$ on each side. A like amount should be used if possible at column splices (page 276 : 4). When plates must be inserted between angles it is desirable to have the space between the angles $\frac{1}{8}''$ more than the plate thickness; when this is not feasible, care should be taken that no shop rivets are placed in either leg of the angles which will prevent their being spread sufficiently to allow the plates to enter. If such rivets are required they should be left to be driven in the field (Fig. 135). Often splice plates and connection angles may be held in position by one or more shop rivets (Fig. 133), but if erection is thus made more difficult it is better to omit all shop rivets and to ship the pieces bolted (Fig. 135).

5. **Driving Clearance.**—Both shop and field rivets are preferably driven by machine as explained on page 30 : 4, and if possible rivets should be so located that the machines can be used. A careless draftsman sometimes locates rivets which can be driven only with the greatest difficulty, if at all.

A common mistake among novices is to space the rivets in the cover plates of columns or girders independently of the stiffening angles on the web; as a result the outstanding legs of the stiffeners often interfere with the driving of the rivets in the cover plates.

* See also Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

In order that machines may be moved into the proper position for driving rivets, sufficient driving clearance must be provided between the rivet heads and any projecting parts to allow for the dies which form the heads. The amount of driving clearance required under different conditions is given in the tables at the bottom of page 304.

1. **Other Erection Considerations.** — (*a*) Too much care cannot be taken to insure the proper erection of each member at the site. Not only should the holes of one member match exactly the holes of a connecting member, but the erectors should be able to put every member into position without interference and without undue labor. The difficulty of attaining this end increases in proportion to the size of a contract and to the number of detailers and checkers who work upon it simultaneously. (*b*) Field rivets should be so located that they can be driven when the members are in position; bent plates and projecting parts of other members are liable to interfere. (*c*) Any mistake which leads to cutting or drilling in the field is very expensive, partly because of the lack of facilities but more especially because of the number of skilled workmen who are delayed during the investigation which necessarily precedes any alteration. If a member must be returned to the shop for changes, or if a new piece must be made to replace a member, the delay in erection is often very costly.

An actual blunder may be cited to show the importance of studying the structure as a whole. Two girders at right angles to each other were to be connected to the same column, one to the web and the other to the flange. Each connection would be correct if used independently or if one of the girders extended in the opposite direction, but the detailer and the checker both overlooked the fact that the two girders would intersect and hence could not be in position at the same time. It was necessary to cut one girder short and make it frame into the other; also to strengthen the other column connection to support the combined load.

(*d*) Details should be arranged to facilitate erection whenever possible, but this is especially important in replacements, such as office and loft buildings, and railway bridges, so that the old structures may be left intact until the last possible moment. Special types of connections are often used for this class of work in order to reduce the number of field rivets (page 89 : 2). (*e*) The draftsman should be familiar with common methods of erection * in order to anticipate special requirements for which provision should be made on the drawings. Rivet heads which protrude far enough to prevent swinging a member into position should be flattened or countersunk (page 40 : 6), or else left to be driven in the field. "Hand holes" may have to be bored through solid web plates to give access to the inside of box sections for driving rivets. Similarly, shop rivets may be omitted from lattice bars or tie plates so that they may be removed temporarily (Fig. 129). Single field rivets at the intersections of the diagonals of vertical bents may require special stagings for the riveters. Such rivets are very expensive and they should only be used when unavoidable. (*f*) When the position in which a member is to be erected cannot be readily determined from the member or from the erection diagrams, one end of the member should be marked N., S., E., or W., or otherwise, to show the proper position. Shopmen should always place a member right side up, before painting the shipping mark on the side. This system will prevent the erection of a member upside down. In some cases it may seem desirable to make the spacing of one or two rivets different at the two ends to prevent interchange. Similar precautions should be taken to avoid mistakes in assembling connection angles or other parts of members which may cause trouble in erection. (*g*) When two or more members are to be supported by the same field rivets, they must all be erected before the rivets are driven. Such conditions are usually apparent from the erection diagrams, but the draftsman should make sure that any unusual connections are noted or indicated on the diagram in such a manner that the erector will not drive any rivets prematurely. (*h*) Bridge details should in general be arranged so that the trusses or girders may be completely erected before the floor system is inserted or, conversely, so that the floor system can be completely erected before the trusses are put in position. (*i*) Notes should appear on the erection diagram drawing attention to all drilling and cutting which must be done in the field, such as holes in existing structures for the connection of new members. This shows the erectors that such work is expected of them so that no claim for extra remuneration can be made.

* See footnote, page 20.

CHAPTER XV

LAYOUTS

SYNOPSIS: There are three common types of layouts in which the graphic method of determining certain dimensions may be used to better advantage than numerical computation.

1. **A "layout"** is a preliminary drawing made for the purpose of scaling distances which cannot be obtained so easily in any other way. Layouts are used chiefly for determining the best shape and size of connection plates for members which meet at oblique angles, with due regard to rivet spacing, edge distance, and clearance. A layout is usually made on a separate sheet, and having served its purpose it is generally discarded just as a calculation would be; such a drawing need not be made so complete as a working drawing, but it should be drawn much more carefully to scale. In order that distances may be scaled with accuracy a layout should be drawn to a comparatively large scale; for most work a scale of $1\frac{1}{2}'' = 1'$ is satisfactory, but $3'' = 1'$ or $1'' = 1'$ are often used. A separate layout is usually made for each gusset plate or bent plate connection, although similar layouts may often be combined. The term "layout" is also applied to a preliminary drawing which may afterward be completed for use as a working drawing. Thus, the drawing of a truss, for example, may be carried far enough for the draftsman to scale the lengths of all members with sufficient accuracy to enable him to order the material, and later this layout may be completed to serve as a working drawing.

2. **When Used.** — A layout is necessary only when the axes of the members intersect at angles other than 90°, for if members meet at right angles all dimensions may be determined easily by addition and subtraction. Layouts are commonly used for practically all skew connections, and for the gusset plates of trusses and various forms of bracing. When working drawings are made to a scale of $1'' = 1'$ separate layouts are not usually required because the sizes of the gusset plates may be determined with sufficient accuracy from the drawings.

3. **A simple layout,** composed of a few limiting lines such as edges and ends of members, often suffices to give the experienced draftsman the desired information, but a novice can often save time by putting in extra lines to show the conditions more clearly. A draftsman should make a layout more complete if it is to be used also by someone else. If an elaborate layout is made for use in ordering material, particularly when conditions are rather unusual, this layout should be preserved for the use of the detailer and the checker. It need be carried only far enough at first to serve the purpose of the man who orders the material, when it may be handed to the detailer to be completed for his use. Most checkers prefer to make new layouts in order to obtain more positive checks, particularly if inaccuracies in plotting will seriously affect the results.

4. There are **three common types** of layout which will serve for illustration, viz.: gusset plates, lateral plates, and bent plates. The first two terms are often used interchangeably, but for convenience they will be treated as separate types with this distinction: gusset plates are used to connect main members of trusses or other members carrying considerable stress when it is important that their lines of action meet in a common point; lateral plates are used to connect the members of bracing systems or light latticed girders in which a single intersection is less important so that auxiliary working points may be used for convenience.

1. As an example of the first type there will be given the general **method of procedure for** making the layout of **a gusset plate** which connects two web members of a roof truss to the bottom chord, each member being composed of two angles. See Fig. 76. I. Determine the slopes of the diagonal members as explained in the next paragraph (see also page 115:2). II. Lay down the working lines (usually the rivet lines, see page 115 : 2), of all the intersecting members to the proper slopes, using a scale of $1\frac{1}{2}'' = 1'$ or $3'' = 1'$. III. Plot the limiting lines (outside edges) of the angles, using the proper gages (page 68 : 5), and showing the backs of the angles on the proper sides. IV. Draw lines to show the desired clearance c (page 72 : 2). V. Cut each diagonal angle normal to its axis so that the nearest corner will fall in the clearance line just plotted. Make sure that ample clearance is left between the diagonals. VI. Place a rivet at the desired edge distance e (page 69 : 3) from the end of each diagonal angle; since the distances from these rivets to the working point are to be dimensioned on the working drawing, it is usually preferable to express them to the nearest $\frac{1}{4}''$, either the amount of clearance or the edge distance being changed, if necessary, to accomplish this result. VII. Lay off the proper number of rivets (page 231 : 1) in each diagonal member at the desired distance r apart; this distance is generally made equal to the usual or the preferred minimum spacing given in the table on page 305, so that the resulting plate is not made unnecessarily large. VIII. Space the proper number of rivets in the chord member; if the chord is continuous, rivets need be provided only for the difference in the stress on opposite sides of the plate (see page 233 : 2). These rivets can often be spread more than those in the diagonals without increasing the size of the plate; the outer rivets should preferably be placed at the distance e from the edges of the plate; this means that they can often be projected down from the last rivets in the diagonals, although in that case the shape of the plate must be anticipated and perhaps the size must be determined (see IX below) before the rivets can be located. If feasible one rivet should be placed at the working point where it is most effective, but care should be taken that no space exceeds the maximum allowed (page 69 : 1). IX. Draw in the edges of the gusset plate with due regard to the following points: (1) allow ample edge distance e from each rivet to the *nearest* edge of the plate; (2) leave no corners of the plate projecting beyond the angles, *i.e.*, each vertex should be hidden by the angles; a corner which falls behind a *single* angle should preferably be made to come on one edge of the angle for the sake of appearance, but if between two angles this does not matter; (3) reduce the number of cuts to a minimum, for each cut increases the cost of the plate; (4) avoid cuts with reëntrant angles, for they cannot be sheared; they must be cut by punching a series of connected holes and then chipping off the remaining projections between holes with a pneumatic chisel (page 14); this operation is reserved for very exceptional use (see *SP*1, Fig. 128); (5) make two edges of the plate parallel and a whole number of inches apart if possible, so that the plate may be cut from one of standard width (page 43 : 3); (6) cut across the full width of the plate if possible, so that a number of similar plates may be cut without waste from a long plate, by alternating the cuts as shown in Fig. 44; (7) the width of the plate is usually the shorter dimension but this may be changed if desired on account of (5), (6), or (8); (8) use as few different widths of plates as practicable on one drawing, and preferably on one contract; they may then be ordered or taken from stock to better advantage; (9) the nominal length of the plate as billed on the drawing is the extreme dimension at right angles to the width, *i.e.*, the dimensions of the including rectangle are given; this length is preferably expressed to the nearest $\frac{1}{4}''$ but eighths are used; in ordering material in multiple lengths advantage should be taken of any gain which may result from (6), for if the extreme lengths were added together more material than is required would be ordered.

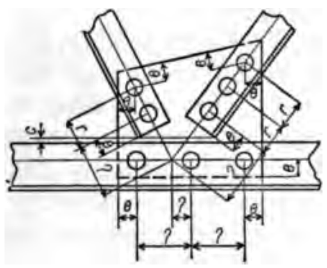


Fig. 76.

2. **The calculation of the slope** or the bevel of one line with reference to another line is based upon similar right triangles. As explained on page 50 : 7, the slope is represented by the tangent of the angle reduced to a base of 12″, although the actual value of the angle is seldom used. A system of working lines is usually laid down in such a way that the rectangular coördinates of any intersection measured from any other

intersection may be easily determined. Often these coördinates are dimensioned on the drawing, or else they may be found from the proportion of similar triangles. These coördinates form the two legs of a right triangle, in which the slope and often the length of the hypotenuse are required. The tangent of the smaller angle of the triangle is found by dividing the length of the shorter leg by the length of the longer leg; if this division is effected by means of a table of logarithms arranged by feet, inches, and fractions of inches, the resulting tangent will be expressed as desired in inches and fractions of inches (to the nearest sixteenth), corresponding to a base of a unit foot (12 inches). If the length of the hypotenuse is required also, it should be calculated at the same time as the slope by means of parallel tables of logarithms and squares* as illustrated by the following problem. In Fig. 148 the holes along the diagonal edges of *M* 1 and *M* 2 are referred to the working line of the supporting angle. The given coördinates are 3′-7¼″ and 10′-2½″. The corresponding slope and length of the diagonal as shown on the drawing are obtained as follows:

Length	Logarithm	Square
3′-7¼″	0.55680	12.9900
10′-2½″	1.00895	104.2101
	(difference) 9.54785	(sum) 117.2001
	slope = 4¼″ in 12″	length = 10′-9$\frac{15}{16}$″

Note that the tables are expressed to thirty-seconds to facilitate the selection to the *nearest* sixteenth of the desired slope or length from the corresponding logarithm or square.

1. Lateral plates are commonly used for lateral bracing in bridges, diagonal bracing in buildings, in place of gusset plates in light latticed girders, or wherever the stresses are so small that a slight deviation from a single point may be made in the intersection of the lines of action of the members. An auxiliary system of working lines is drawn through the end rivets of the diagonals. The two principal advantages of this type of plate connection are (1) that the clearance between members can be made more nearly equal with a corresponding reduction in the size of the plate, and (2) that a comparatively simple layout may be made; in fact the desired information may often be obtained easily without a layout. This type of connection is illustrated by a plate in a latticed girder, as shown in Fig. 77. I. The rectangular coördinates which locate the end rivets of the diagonals are usually made the same for the upper and the lower ends of the diagonals so that the plates may be made alike or similar. II. For most systems of diagonal bracing in which the leg of the diagonal is 3″ or more, the rivets are placed on the center line of the leg in order to make all clearances nearly equal; for latticed girders and small diagonals the standard gage is used, and thus when provision is made for the proper clearance for the corner at the *back* of an angle, a larger clearance will be left on the opposite side, since the gage is larger than the remaining distance. III. The slope of the diagonals cannot be calculated until the end rivets are located, and hence the position of the corners of the angles relative to the end rivets is unknown; it is customary to allow sufficient space in any direction for the maximum distance from the rivet to the farther corner of the angle. This distance is the hypotenuse of a small right triangle one leg of which is the gage (page 68 : 5), and the other the edge distance *e* (page 69 : 3); this may be found from the diagram on page 313, or from a full-sized layout.† IV. The vertical distance (usually to the nearest ⅛″) from the end rivets in the diagonals to the rivet line in the chord may now be found by adding three component parts, (1) the edge distance (leg minus gage) of the chord angle, (2) the desired clearance (page 72 : 2), and (3) the diagonal distance found in III. Similarly the horizontal distance between the end rivets of the two diagonals is twice the distance found in III, plus the clearance; in latticed girders this distance may be

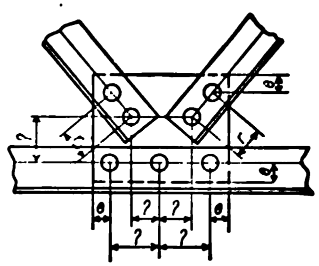


Fig. 77.

* A copy of Smoley's "Parallel Tables of Logarithms and Squares," McGraw-Hill Book Co., Inc., New York, should be included in the equipment of every structural draftsman.

† A convenient table for use in spacing the end rivets in diagonals is given by E. Feldman in *The Engineering News-Record*, Jan. 16, 1919.

changed slightly as explained on page 109 : 1. V. Either of two methods may be used for the remainder of the problem; (1) a layout may now be made of the working points already established and the remaining distances and the size of the plate may be determined graphically as outlined for gusset plates in steps VII, VIII, and IX, page 76 : 1; or (2) the required data may be found without a complete layout, as follows: VI. From the proper number of spaces r in each diagonal find the corresponding horizontal and vertical coördinates; these may be found (1) from simple layouts drawn separately; (2) from the main working drawing upon which the working lines have been plotted to scale; the diagonal distances may be scaled along the proper working lines and the corresponding components measured without drawing any extra lines; or (3) by placing a straight-edge in the proper position on the diagram on page 313, and then reading the desired coördinates. VII. The dimensions of most lateral plates may be found arithmetically by the proper combination of the edge distances and the distances found in IV and VI; when diagonal cuts are used the positions of the corners of the plate may be determined from a few lightly penciled lines on the main drawing (see VI (2) above).

1. In **bent-plate work,** a layout is often necessary in order to determine the shape and the size of the connecting plate. Such a layout is made of the *developed* plate, i.e., the plate before it is bent. The bend line is plotted and parts of the members to be connected are drawn in proper relation to a definite point in this bend line, not in their actual relation to each other. The rivets and holes may be laid out to the best advantage in each member and then the plate may be formed around these rivets and holes according to the suggestions given under IX, page 76 : 1. Such layouts are used for special skew work beyond the range of this book; * they are not required for ordinary bent plate work in which the holes and the edges of the plate are laid off either parallel or perpendicular to the line of bend, and in which the dimensions for the plate are determined numerically. See Figs. 78, 93, and 149. When bent plates connect to web plates, it is usually better to refer dimensions to the center lines of webs as working lines instead of to the faces of the bent plates, as shown in the above figures. The drawing is thus simplified although a corresponding burden is imposed upon the templet maker; but the method is more direct and there are fewer sources of error. Dimensions should be so given that they truly represent the desired measurements; only distances which are parallel to the line of bend can be dimensioned in an oblique view of a plate, i.e., in the portion of a plate which is not parallel to the plane of the drawing. See page 141: 4.

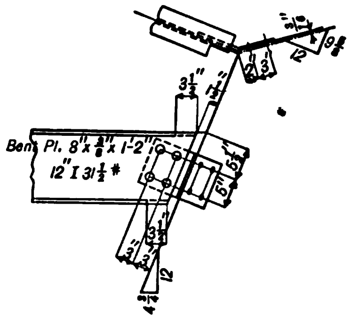


Fig. 78.

Care should be taken to place rivets far enough from the line of bend so that they can be driven after the plate is bent. If only one bent plate is used it is better to place it on the obtuse-angle side rather than on the acute-angle side. Bent angles may sometimes be used instead of plates if they are not bent more than about 3″ in 12″; the line of bend is likely to be at the edge of the fillet instead of at the vertex, so that larger bends are unsatisfactory.

* For illustrations see the author's "Hip and Valley Rafters," John Wiley and Sons, Inc., New York.

CHAPTER XVI

MARKING SYSTEMS

SYNOPSIS: An identification mark is assigned to each member, and this mark is used on the drawing, on different lists and diagrams, on the templets, and on the steel. Similar marks may be used on the component parts of a member for use in the shop.

1. There are **two kinds of marks** in common use in structural work, the one "Assembling Marks" or "Piece Marks," and the other "Shipping Marks" or "Erection Marks." Assembling Marks are used on the small component parts of a member to facilitate their fabrication in the shop. Shipping Marks are used on the completed members such as beams, girders, or sections of trusses, as they are shipped from the shop to the site, to serve for identification in the drafting room, in the order office, in the shop, in shipment, and in erection.

ASSEMBLING MARKS

2. **The assembling marks** usually originate in the drafting room, where they are put on the drawings; in some companies they originate in the templet shop, where they are indicated on the blueprints with colored pencil, and then the prints are passed on to the structural shop. In either case, the marks are painted on the templets, and also, after the holes have been laid out from the templets, the marks are painted on the steel to aid the fitters in assembling the parts. The use of assembling marks enables the templet maker to make a single templet for a detail which occurs on many different sheets, although this may be done to a lesser degree when assembling marks are not used. No further mention will be made of any system of assembling marks which may be arranged between the templet makers and the fitters, for it has no bearing upon the drafting room.

3. **When Used.** — The draftsman should ordinarily avoid the use of assembling marks unless some apparent benefit will be derived. The benefit to the draftsman will depend largely upon the system used, but the benefit to the shop will be in proportion to the number of times the same detail is repeated, especially if on different sheets. In some instances, the use of assembling marks may increase the cost of the drawings, but this may be offset by the reduced cost of the shop work, particularly in large contracts. Assembling marks are not given to the main component parts of a member, but only to those details which occur more than once on the same or different sheets. When there are only a few details of the same nature on a sheet, there is less need of assembling marks than when many similar pieces occur on the same member to confuse the fitters. Moreover, the sheet number and the shipping mark (page 80 : 6) are painted on each piece, and frequently the assembling marks are superfluous. Only large contracts with many details require the use of assembling marks. Occasionally, as in plate girder work, for example, it is convenient for the draftsman to use assembling marks on some parts, as the stiffeners or fillers, in order to save repeating the dimensions and sizes, even though marks are not used on the remainder of the contract (see Fig. 102 or 103).

4. Many structural companies use systems of assembling marks which differ from each other in minor details, but the principles underlying most of them are essentially the same. The assembling mark immediately follows the billed size of the piece, and consists of **three parts,** viz: (1) a characteristic letter which indicates the nature of the piece, (2) a specific letter (or number) which distinguishes the piece from others of the same nature, and (3) the number of the sheet where the detail is

first shown. The letters should be lower case to distinguish the assembling marks from the shipping marks. The use of assembling marks is illustrated by many of the drawings of this book, as for example Figs. 102 and 116.

1. **The first letter** indicates the style of the detail and should preferably be suggestive, as for example: — b = bottom seat angles for beam connections, f = fillers, m = miscellaneous angles, p = miscellaneous plates, s = stiffening angles, t = top angles for beam connections, and w = light web members. Other letters are used for various pieces, as a = base or cap angles of columns, c = cap plates, base plates or splice plates, h = bent plates or angles, y = lattice bars, etc.; these are not so significant as the first group, and accordingly are not so generally used since each company adopts its own code. Some companies even differentiate between stiffeners which are fitted at both ends and those which are fitted at one end only, or between fillers with two rows of holes and those with only one.

2. **The second letter** (or number in some systems), is used to show the difference between similar details on a sheet. For instance, the end stiffeners of a plate girder might be marked *sa*, the next pair *sb*, and other different ones *sc*, etc., double letters being used when the alphabet is exhausted, as *saa*, *sab*, *sac*, etc. The letters i, l, o, q, and u are usually omitted because they are not readily distinguishable.

3. **The sheet number** refers to the sheet where the piece is first detailed. *On this sheet* the number after the letters of the mark is often omitted; it may be understood that the men in the shop are to consider the number in the corner of the sheet to be a part of each mark, unless another number is given, and much time may thus be saved. For illustration, if a filler was first detailed on sheet 6, where it might be marked simply *fd*, the shopmen would mark the templet and the steel *fd* 6; if a filler exactly like it occurred on sheet 8, it would be marked on the drawing *fd* 6 instead of *fd*, to show that it was detailed on sheet 6 and to prevent the shopmen from marking it *fd* 8, as they would if the sheet number were omitted. For the convenience of all concerned, the piece should be completely dimensioned and billed *once* on *each* sheet, but in all other positions on a sheet the dimensions and the billing may be omitted. The assembling mark is placed near the detail in each position. The dimensions for all field rivets are sometimes repeated to simplify comparison with those of connecting pieces.

4. All pieces which are identical should be given the same mark, and, conversely, pieces should have different marks if they are not interchangeable. Not all companies indicate **rights and lefts** (page 81 : 2), in assembling marks, but expect the templet maker to distinguish them by means of the drawing. It seems preferable, however, for the draftsman to complete the system if it is to be used at all, and to indicate rights and lefts on the drawing (see Fig. 104).

5. Sometimes a **summary** of assembling marks is placed in the corner of each sheet giving the number of pieces of each mark required for the members to be made from that sheet. A number is added to the sheet where each piece is first detailed, to show the total number required for the entire contract. While this method is of convenience to the templet maker, it is not to be recommended unless it is possible to complete all drawings of similar members before any are sent to the shop. Otherwise, later drawings will refer to sheets already in the shop with a corresponding change in the totals; revised prints must then be issued — a practice which should be reserved for unforeseen corrections or alterations.

SHIPPING MARKS

6. **The shipping marks** should be clearly shown on the detailed drawings. From the time the draftsman determines the shipping mark of a member, until the member is in its final position in the structure, it is known by this mark. The mark appears on the drawing, on the order bills, the shop bills, the shipping bills, and the rivet lists, and it is painted on the templets and on the individual pieces of steel of which the member is composed. The mark is preserved when the completed member is painted before shipment, and it serves an important function during erection in enabling the erector to place the member in the proper position, as indicated by a similar mark placed on the erection diagrams which are prepared by the draftsman.

7. In general a shipping mark is composed of **two parts,** viz: a characteristic letter or letters and a specific number, as *S* 14, or *LG* 2. Capital letters are used to distinguish clearly shipping marks from assem-

bling marks (page 79 : 4). As far as possible the letters should be suggestive of the type of member, as for example, C = columns, G = girders, EP = end post, etc. For a list of letters commonly used for different members, see page 324. A special system of marks is used for office-building construction in order to indicate the floor numbers, as explained on page 81 : 5. Truss members are usually marked according to a system of panel point letters, as explained on page 82 : 1.

1. Shipping marks should be **marked conspicuously** on the drawing either just below the drawing (Fig. 87) or at the right of the sheet above the title (Fig.100). When several different members are shown on the same sheet the marks may be tabulated in a "Required List" above the title (Fig. 135); when these members are represented by different drawings on the sheet, each drawing should bear the corresponding shipping marks, as shown in Fig. 140.

2. **Rights and Lefts.** — All members which are identical should bear the same shipping mark (except in office buildings, page 81 : 5), and conversely no two members should be marked the same unless they are interchangeable. When pieces are *exactly* opposite they may be marked "Right" and "Left," one drawing serving for both. The drawing should be made for the member marked "Right"; the "Left" member is then made as if the drawing were reversed; the marks should be placed on the erection diagram accordingly. No indication of rights and lefts need be made on the main part of the drawing, the only difference being made in the list of members required, where the rights and lefts are distinguished by adding the capital letters R or L to the shipping mark, as in B 4, Fig. 85, and C 5, Fig. 137. Before marking pieces right and left, the draftsman should satisfy himself that the pieces are really opposite, and that there are no other differences. The novice frequently imagines that two members are opposite when in reality they are interchangeable if inverted or turned end for end. A conception of rights and lefts may be gained from Fig. 81. If all the details are reversed about any *one* of the three axes of symmetry a left is obtained, but if they are reversed about any *two* axes, one reversion counteracts the other and the piece remains unchanged. If a member were placed in front of a mirror the *right* would be represented by the real member and the *left* by the reflected image.

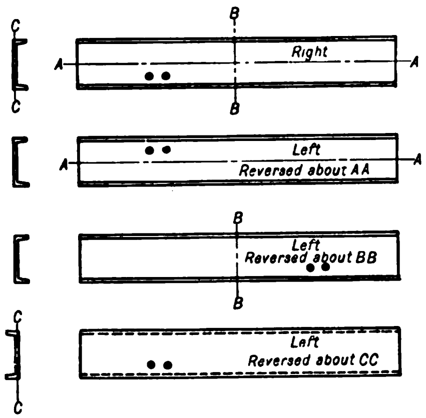


Fig. 81. Rights and Lefts.

3. **Members Combined.** — One drawing may be made to serve for several different members if the differences are properly indicated or noted, as explained on page 53 : 4. Members which are marked rights and lefts may be combined with other members on a drawing, but the details should be shown just as if only the rights were combined. It is unnecessary to indicate "R" or "L" in the various notes on the drawing, but simply the remainder of the mark, since all notes must necessarily apply to both the "R" and the "L." For example, in Fig. 135, the required list calls for columns $C\ 1^R$ and $C\ 1^L$, but each individual note on the drawing refers to C 1 only, with the understanding that it applies to both, and that $C\ 1^R$ is made like the drawing, and $C\ 1^L$ is made opposite. No member should be marked "Left" unless there is a corresponding "Right." (See next paragraph.)

4. **Opposites.** — Two members which cannot be marked R and L because they are not *exactly* opposite may be so nearly opposite that they can be combined on the same drawing to advantage. They must be given different marks and in the required list the mark of one must be followed by the word "Opposite," as in R 22, Fig. 93. Such a member would be made as if the drawing were reversed, in much the same manner as a "left" is made, but in accordance with special notes or dimensions. All details which apply only to the member marked "Opposite" should be drawn in the proper relation to all other parts, subject to reversion with the rest when the member is made.

5. A special system of marks is usually adopted in **office-building construction** in order to distinguish between members which are erected at different stages. Usually the columns are erected in two-story lengths and the two corresponding tiers of beams and girders are erected at the same time; all the derricks are then raised two stories and the

process is repeated. Members should be shipped approximately in the order of erection; this is usually necessary because of the lack of storage facilities, and is desirable because it simplifies erection. All identical beams and girders in any one floor bear the same mark; identical beams and girders in different floors may be combined on the same sketch and have the same specific *number*, but they must bear different *floor marks* (see *C* 17, *D* 17, *E* 17, Fig. 92). Office-building columns are numbered consecutively according to some definite system for convenience in finding a particular number; thus no two columns bear the same mark even though they may be interchangeable. The columns are numbered the same on all plans so that each column bears the same specific number as the column directly above or below it but the floor marks are different. Two methods of marking office-building members are in common use. **First Method.** Each beam should bear the character "#" for "Number" (instead of a letter), followed by a specific number and by the floor number, thus: #25 — *3rd FL.*, or #3 — *ROOF*. Each girder should bear the letter *G* followed by a specific number and by the floor number, thus: *G* 2 — *BASMT.*, or *G* 5 — *4th FL.* Each column should bear the abbreviation "*COL.*," and a specific number, followed in parentheses by the numbers of the floors between which it extends, thus: *COL.* 11 (0–2), or *COL.* 19 (10–*ROOF*). **Second Method.** A capital letter is assigned to each floor or tier of beams, as *A* = basement, *B* = first floor, *C* = second floor, and so on up to *R* = roof; the letters *I*, *O*, and *Q* are omitted to avoid confusion with similar letters or figures. Each beam should bear a floor letter followed by a specific number, thus: *D* 25, or *R* 3 (see also Fig. 87). Each girder should bear the letter *G* followed by a specific number and by the floor letter or number, thus: *G* 2 — *A-TIER*, or *G* 5 — *4th FL.* Each column should bear the letters of the tiers of beams which it supports, followed by a specific number, thus: *AB* 11, or *MR* 19 (see also Fig. 133). If another story were inserted between tiers *M* and *R*, the top section would become *R* 19 or preferably *MNR* 19, depending upon the length.

1. Truss members are marked according to the letters of the panel points or apices between which they extend. In **bridge trusses** the upper apices are marked *U* 1, *U* 2, etc., and the lower apices are marked *L* 0, *L* 1, *L* 2, etc.; these marks are so arranged that *L* 1 is directly under *U* 1, etc., as shown in the sketch on page 324. Web members are marked with the panel marks at their ends, thus: *L* 2–*U* 1, or *L* 2–*U* 2 (Figs. 126 and 129). Chord members need not have the letter *L* or *U* repeated, thus: *L* 2–3, or *U* 1–3 (Figs. 125 and 124). End posts are usually marked *EP* (instead of *L* 0–*U* 1), (Figs. 122 and 127). For the sake of uniformity, drawings are made for the members in the *left-hand* half of the *far* truss and the members should be marked right and left accordingly.

2. A **roof truss** is usually erected as a whole in order to save falsework. A small truss may be shipped completely riveted, but a larger truss must be shipped in sections; these sections are assembled and riveted together on the ground and the entire truss is then lifted into position. A whole truss is marked on the erection diagrams with the letter *T* followed by a specific number, but unless the truss is shipped completely assembled, each member or truss section must bear a separate shipping mark. An assembling diagram is shown on the sheet where the truss is detailed (Fig. 116), and sometimes this is duplicated on a smaller sheet more convenient for the erector. The apices are lettered as illustrated in the sketch on page 324, the letters being the same in both halves with the exception of *A* and *X*. When one half of a truss is shipped in one section it is marked either *AH* or *XH* followed by a specific number, the half trusses on one side of the building being marked *AH* to differentiate them from those on the other side which are marked *XH*, thus: *AH* 2^R or *XH* 3 (Fig. 116). When smaller sections than one-half of a truss are shipped they are marked by the three letters at the extremities followed by a specific number, thus: *ACG* 3, or *CEF* 2. When single members are shipped separately they are marked with the letter at their ends, followed by a specific number, thus: *HH* 1 or *FL* 2 (Fig. 116).

3. Straight **tie rods** and sag rods may be identified by their lengths. On the drawings and erection diagrams these lengths are expressed in inches inscribed in a circle to avoid confusion with other marks. It is unnecessary to paint the number on each individual rod. Bent rods, and main bracing rods are marked in the usual manner with an *X* followed by a specific number.

4. Special **direction marks** may be added to members in order to facilitate proper erection, as explained in (*f*), page 74 : 1.

CHAPTER XVII

BEAMS

SYNOPSIS: In the preceding chapters are given the more general fundamental principles of drawing. These are followed by chapters in which are given more specific information applicable to the drawing of the more common types of members. In this chapter are given the types of connections, the methods of dimensioning, and other conventions and practical points peculiar to I-beams and channels.

1. A beam is a member which resists flexure or cross bending. Usually it is placed in a horizontal position and is subjected to vertical loads. **A simple beam** rests upon two supports and all its loads are applied between the supports. **A cantilever beam** receives part or all of its loads upon the portion of the beam which extends beyond the supports. A cantilever beam may rest upon two supports and extend beyond one or both, or it may be fixed at one end by a masonry wall, or by other means, and be unsupported at the other end. **A continuous beam** rests upon more than two supports and its use should be avoided if practicable. A simple beam is encountered in practice more frequently than any other structural member.

2. Generally speaking, a beam is composed of a **single piece**, exclusive of details, and is usually of wood, of steel, or of concrete. In steel construction, a member which is made of more than one main piece but which acts like a beam, is termed a girder, see Chapter XVIII, page 95.

3. The forms of steel beams which are most commonly used are the **I-beam** and the **channel.** Since I-beams are frequently called simply "beams," care should be taken to avoid ambiguity between the general term which applies either to I-beams or to channels, and the specific term which refers to I-beams only. In this book the term "beams" includes both I-beams and channels.

4. In order to reduce the cost of making structural drawings for beam work, the details of which are usually similar and comparatively simple, most companies furnish **printed forms.*** Upon these forms are outlined I-beams or channels, some with the top and bottom views, and some without. A few dimension lines are also printed, the others being added by the draftsmen as required. The use of these forms allows only one size of sketch regardless of the actual dimensions of the beams to be drawn, and consequently the drawings cannot be drawn to scale. It is best, if practicable, to plot the *details* according to the scale which most closely corresponds to the depth of the beam, but to estimate the distances *between* the details so that they are approximately proportional to the total length. A complete sketch should be drawn to scale when the number of details or the complexity of special connections warrants it; a blank sheet of the same size, and with the same printed headings is provided for this purpose.

5. **Beams are supported** either by masonry walls, or else by connections to other beams, to girders, trusses, columns, etc. For any given type of connection the details used by the different companies are quite similar.

6. **Standard Connection Angles.** — The beams of mill buildings and similar structures are connected to each other so generally by means of

* In order to enable the student to become familiar with blanks similar to those used in practice, the author has prepared several forms suitable for use in any university, copies of which may be obtained from the publishers, John Wiley and Sons, Inc., New York. The drawings of this chapter, before one-half reduction, were made on these forms.

angles riveted in the shop to the ends of the supported beams that most companies have adopted standard connection angles. In 1912 the American Bridge Company adopted the connection angles shown on pages 298 and 300. These differ from the older forms shown on pages 299 and 301 in the size of the angles and in the number and the spacing of the rivets, but they were adopted only after sufficient tests had been made to justify their use. They have since been incorporated in the handbook of the Carnegie Steel Company, and are used in the drawings of this book. The older forms are distinguished in the tables as "Lackawanna" angles although similar standards are shown in the handbooks of the Cambria, Phoenix, Pennsylvania, and Jones & Laughlin Steel Companies, and others, and formerly were used by the American Bridge Company. Connection angles for Bethlehem beams are shown on page 302. In the new form mentioned above the vertical spacing is 3″ without exception, while in the old form both 2½″ and 3″ are used. On account of the different web thicknesses either the gage in the angles or else the distance between the holes must vary. In the old form a standard gage is used, with a variable distance center to center of holes, while in the new form the "constant dimension system" is employed, i.e., the distance between holes is maintained 5½″ regardless of the resulting gages. The constant dimension system is sometimes used with the old form angles as well, both types being shown in the tables on pages 299 and 301. The constant dimension system is recommended, for it simplifies the details, especially when beams of different web thicknesses frame on opposite sides of a supporting web with rivets in common. The system is also well adapted to interdepartmental short-cuts (see below), but it involves a larger number of standard templets. This number is reduced in the plants of some companies by the use only of the gages which are multiples of eighths. Where the tables give values for b in sixteenths these companies use the eighth above on one side of the web, and the eighth below on the other side, thus throwing the beam $\frac{1}{16}$″ off center. In the drawings of this book gages are used as they appear in the tables.

1. Fig. 85 shows an I-beam and a channel detailed according to **three different methods.** The upper modified method is the one adopted in this book, the lower one shows the old form of connection angles,* while the middle one shows the American Bridge Company's method which is greatly simplified by means of interdepartmental understandings.† The students should learn first the more general method, but later, as draftsmen, they must make their drawings conform to the standards of the companies for which they work.

2. In the **modified method** the connection angles should be dimensioned and billed as shown in B 1 and B 4, Fig. 85. The center of each connection should be located from *one* flange of the beam even though the angles are placed centrally on the beam as is usually the case. The holes in the webs to provide for the standard connection angles of other beams are dimensioned in groups, and then the centers of the groups are located vertically and horizontally. Note that the holes of each group are not definitely dimensioned from these centers but it is assumed that the holes are placed symmetrically about the center lines. This is one of a few such assumptions made in structural drafting, most dimensions

* The older forms of connection angles, as shown on pages 299 and 301 or with slight modifications, are used by many companies. The vertical spacing for all but the 18″, 20″, and 24″ I-beams is 2½″. The gage in the outstanding legs is constant, and the distance between holes variable, as shown by the values of a, page 299. Two methods of dimensioning the connections of channels are used, the one giving the total distance center to center of holes as for I-beams but the other giving the distances from the holes to the backs of the channel webs, as shown in B 6, Fig. 85. The values of h from the back of the web to the holes in the angle on the opposite face of the web are given on page 301. With these exceptions the method of detailing is similar to the modified method outlined in this chapter.

† The principle differences between the American Bridge Company's method and the method outlined in this chapter may be summarized as follows: — Channels are preferably drawn with the flanges on the far side to correspond to their position on the rollers in front of the multiple punch; the connection angles are neither billed nor dimensioned; the horizontal distances from center to center of connections are omitted, only the extension figures being used except in complicated work; the distances from the flange to the first holes of standard connections are given instead of the distances to the centers of the groups, because the holes are punched by a multiple punch and no templets are used; the holes for tie rods and for anchors are not dimensioned the former being marked "T" to distinguish them from the beam connections; single angle connections are marked "M" for distinction; beams under 15″ which are coped to beams of the same size are shown coped but are not noted; the overall dimension, the ordered length, the number, the mark, and the size are combined on one of the dimension lines, as shown in B 2 and B 5, Fig. 85.

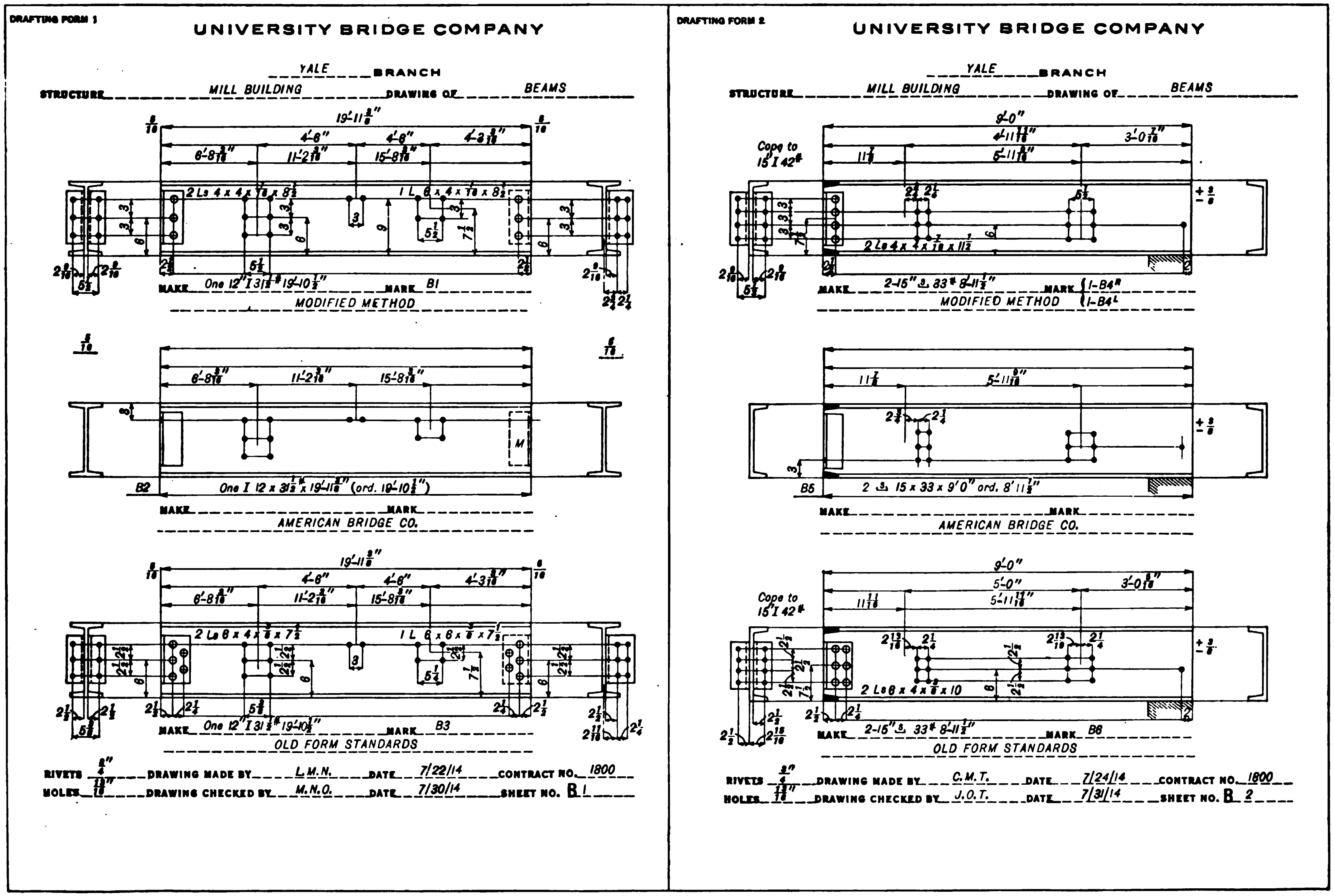


Fig. 85. Comparison of Different Methods of Detailing Beams.

being made more definite to permit of no misunderstanding. Holes other than those for standard connection angles may be dimensioned as in *B* 12, Fig. 87, or *C* 17, Fig. 92. The vertical dimensions should be referred to one flange only, because the actual depth of a beam does not always agree exactly with the nominal depth, on account of the wearing of the rolls or of the spreading of the flanges while they are cooling. Usually they are referred to the bottom flange, except when it is desirable to maintain a uniform elevation of the tops of the beams by referring them to the top flange instead.

1. Where it is impossible to use two connection angles on account of interference with other connections, a single larger angle may be substituted. For the lighter beams, **single angles** are shown in the tables on pages 298 and 300, but for heavier beams similar connections should be used only when they are especially designed to satisfy the given conditions, as discussed on page 234:2. When a single angle is used care should be taken to show by means of full or dashed lines upon which side of the web the angle is to be placed. See *B* 1, Fig. 85.

2. When beams of **different depths** frame on opposite sides of a web, standard connections should be used if possible, even though the angles have to be placed above or below the centers of the beams in order to accommodate the rivets which are in common. If the beams are not in the same vertical plane but too close to permit the use of independent standard connections, special connections must be provided; often two angles with special gages, or a single angle connection mentioned in the preceding paragraph, may be used.

3. **Standard angles are designed** for usual conditions, but they should not be used for short spans or for beams with concentrated loads near the ends unless the number of rivets is found sufficient according to page 234:2. If the number has to be increased, care should be taken that the angles are not made so long that they will interfere with the curved fillets of the beams (page 26:1).

4. Beams are usually **shown full length** even though they are symmetrical about the center lines (compare page 34:5). If the connection angles are alike at both ends of a beam, the duplication of dimensions and also of the end view may be avoided by noting the *right* end "Same as other end," as in *B* 10, Fig. 87. See also page 53:1. The angles should be shown in the web view in order that beams with connection angles at both ends may be clearly distinguished from beams with no angles or beams with angles at one end only. The angles should be billed at each end for the convenience of the billers.

5. The centers of the groups of holes for all intermediate **web connections are located horizontally** in two ways. For the convenience of the men in the drafting room the dimensions are given from center to center of groups and from the ends of the beam to the centers of the end groups (see next paragraph). For the convenience of the men in the shop in spacing the small templets on the steel from one setting of the tape, and also for the use of the inspector, **"extension figures"** are given to indicate the distance from the *left* end of the beam to the center of each group. All of these extension figures may be placed on a single dimension line, provided single arrow heads are so arranged that each dimension extends from the nearest arrow head on the right to the first arrow head on the left that faces the opposite way, i.e., the one at the left end, see *B* 1, Fig. 85. It is not necessary to repeat the dimension at the left end; it is usually given on the line of extension figures. Similarly, it is unnecessary to insert the overall dimension on the line of extension figures, provided it is given elsewhere. Beams which have connection angles on one end only, should be drawn with the angles at the *left* end so that the extension figures extend to the more definite end (compare page 88:1). The center of each group of holes is located on the drawings even though the connection is for a channel. Since the spacing of channels on the erection diagrams is given to the *backs* of the webs and not to the center lines, as for I-beams, it is important that this difference be taken into consideration when the dimensions which locate channel connections are determined. When the web thickness is given in sixteenths on page 300 or 301, the proper value to use for one-half the web thickness may be obtained by subtracting $\frac{1}{16}''$ from the value of *c*.

6. **The length of a beam** with connection angles at *both* ends is the distance from back to back of angles. This is made less than the clear distance between the surfaces of the supporting members in order to allow the proper erection clearance, as explained on page 73:1. In determining this length the draftsman usually obtains the necessary data from an erection diagram upon which the dimensions extend to the

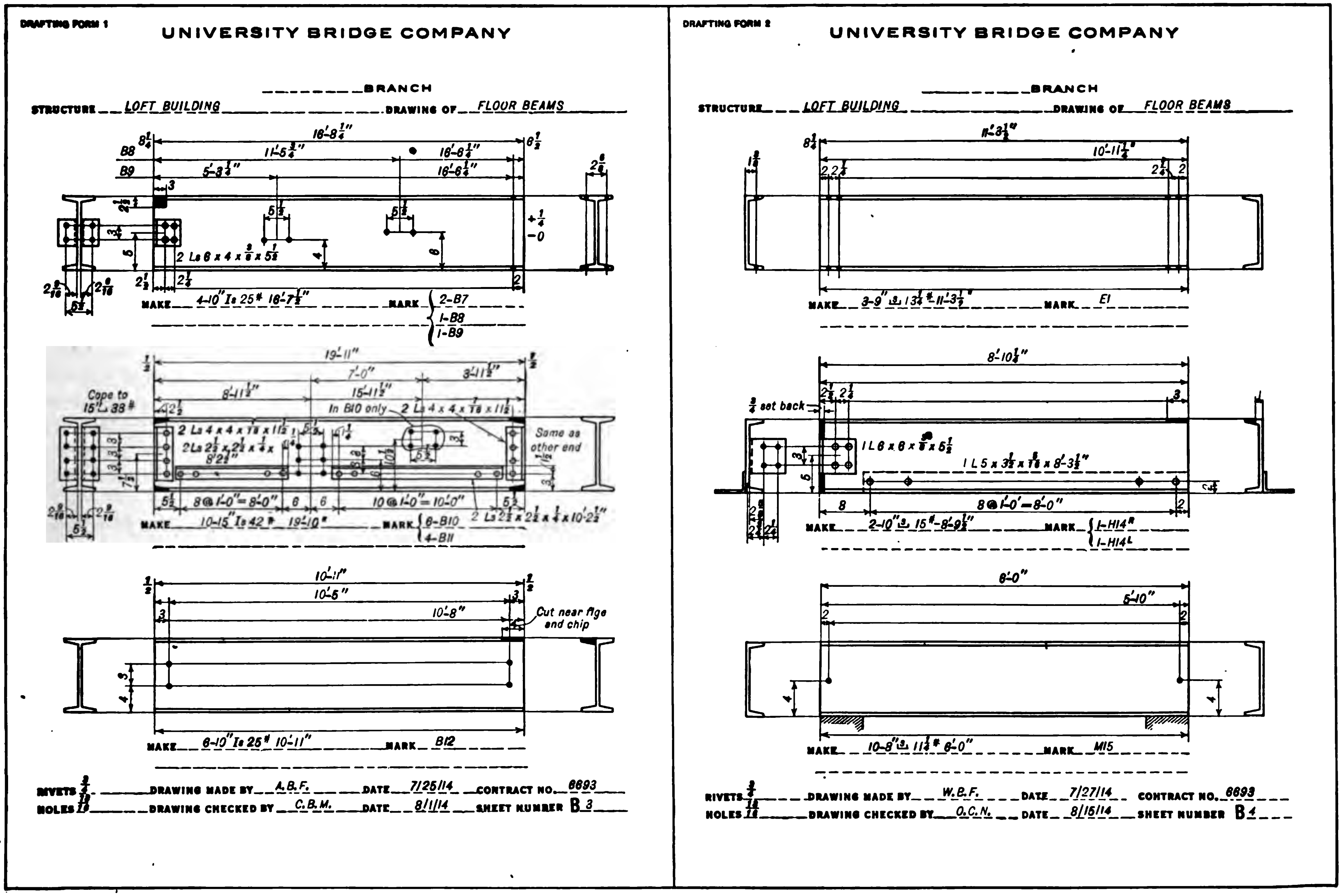


Fig. 87. Typical Beams.

centers of the webs of I-beams, girders, and columns, but to the *backs* of the webs of channels. For the convenience of the detailer and the checker it is well to record at each end of the overall dimension line the distance from the back of the angle to the working line of the supporting member shown on the diagram; these distances may then be used in determining the overall length and also the distance from each end to the first intermediate connection mentioned in the preceding paragraph. In general, the distance from the working line to the back of the angle will be the sum of the erection clearance, usually $\frac{1}{16}''$ (see below), and the distance, if any, from the working line to the surface of the supporting member to which the beam is to be connected. For example, if a beam connects to the web of an I-beam, the distance from the center of the web to the back of the angle will equal *one-half* the web thickness $+ \frac{1}{16}'' = c$, values for which are given on pages 298 and 299. If a beam connects to the web of a channel, the distance will either be $\frac{1}{16}''$, or else the whole web thickness $+ \frac{1}{16}'' = c'$ depending upon whether the beam connects to the outer face (back) or to the inner face of the channel web; values for c' are given on pages 300 and 301. If the main dimension from back to back of angles results in sixteenths, it is preferable to *decrease* it to the nearest eighth (page 34 : 6), and to *increase* the distance from the working line to the back of the angles at one end of the beam to correspond. When connection angles are used at only one end of a beam no erection clearance is required, so the distance from the back of the angles to the working line should be reduced accordingly.

1. Beams are sawed at the rolling mills to the **ordered lengths** while still hot, and each beam must be cut into the desired lengths before the following one leaves the rolls; the lengths cannot, therefore, be measured with great precision and all lengths must be accepted if within the "**mill variation**" of $\frac{3}{8}''$ specified by the steel companies. All beams should be ordered in multiples of $\frac{1}{2}''$ in such lengths that they can be used even though they overrun or underrun $\frac{3}{8}''$; greater allowance is often made to avoid recutting the beams in the shop in case of greater overrun. The material is ordered usually before the detailed drawings are made, but with due consideration of the types of connections to be used. The detailer must make his drawings conform to the ordered lengths if possible. The ordered length is given underneath the detail along with the number of pieces and the size of the beam is billed as indicated on page 44 : 2–3, and as illustrated in the typical drawings of this chapter. When no connection angles are used the overall dimension must manifestly equal the ordered length; but when connection angles are used at one or both ends the overall dimension is assumed to extend to the backs of the angles and the ordered length may differ. Usually the beams are ordered to the nearest $\frac{1}{2}''$ so that each pair of angles will project about $\frac{1}{2}''$ beyond the end of the beam; thus, the ordered length of a beam with angles at one end is about $\frac{1}{2}''$ (from $\frac{3}{8}$ to $\frac{3}{4}$) less than the overall length, while the ordered length of a beam with angles at both ends is about $1''$ (from $\frac{3}{4}$ to $1\frac{1}{8}$) less than the overall length. In order to use printed forms to the best advantage the angles are shown on the drawing *flush* with the ends of the beams, as indeed they may be; the only indication that they are not flush is the discrepancy between the ordered length and the overall length. When conditions require that the end of the beam must not extend within a certain distance of the backs of the angles, the minimum "set back" is indicated as in *H* 14, Fig. 87. Beams are detailed with the understanding that unless noted otherwise an end without connection angles is allowed to vary $\frac{1}{4}''$ from the position indicated on the drawing; other variations may be noted at the end, as for example $\pm\frac{3}{8}$ in *B* 4, Fig. 85, or $^{+\frac{1}{4}}_{-0}$ in *B* 7, Fig. 87; if no variation is allowed the end may be marked ± 0 or else the dimension may be marked "exact." Wall bearing ends (see next paragraph) need not be noted as a rule, for it is evident that greater variation than $\frac{1}{4}''$ is permissible.

2. Beams which are supported by masonry walls must rest on metal **bearing plates** in order that the loads may be distributed over the proper area (page 288 : 2). The size of the plate required for each beam is given in the tables, pages 298 to 302, and the width of the plate (the smaller dimension) indicates the length of the bearing, i.e., the distance which the beam must project on the wall. The bearing plates are not attached to the beams, but the latter are held in place by some form of anchor which is imbedded in the masonry. The anchor most commonly used is the Government anchor, which is a $\frac{3}{4}''$ rod bent as shown on page 316. A hole must be provided for this in the beam 2″ from the end, placed centrally in the web or else on line with

other holes (page 91 : 2). An angle anchor is sometimes used, as shown on page 316. The bearing plates and anchors are shipped separately, and are not shown on the drawing (see page 174 : 1). A wall bearing is indicated conventionally on the drawing by lines which represent masonry, as shown in *B* 4, Fig. 85, and *M* 15, Fig. 87.

1. **Coping.** — When a beam is to frame into the web of an I-beam or the *flange face* of a channel web, it may be necessary to cut away part of one or both of the flanges to prevent interference with the flange of the beam to which the beam connects; the end of the beam thus cut is said to be "coped." A beam may be coped by means of standard dies set in a punch, or by means of an oxy-acetylene flame. Standard coping is shown on the drawing without dimensions, but the size of the connecting beam should be noted, thus: "Cope to 18″ I 55 #"; the *weights* of beams less than 15″ deep may be omitted because no difference is made in the amount of coping for the different weights, thus: "Cope to 12″ ⊔." When beams are drawn on blank sheets the coped portions may be omitted, but when printed forms are used it is not feasible to erase the printed lines and so the coped portions are blackened as in *B* 4, Fig. 85. The blackened portion need not be scaled but for the sake of appearance the distance from the end of the beam to the vertical line drawn between the two lines of the flange should be estimated to be about one-half the width of the flange shown in the end view; the sloping line should be drawn parallel to the sloping line of the end view; i.e., to the standard bevel of 2 in 12, as shown in *B* 4, Fig. 85 and *B* 10, Fig. 87. Care should be taken to show the proper flange blackened, i.e., the top flange when the beams are flush top, the bottom when flush bottom, and both when beams of the same depth are flush both top and bottom. Every cope should be indicated, for a beam will be coped only where distinctly shown; when the connection angles at one end are referred to those at the other end, the note "Same as other end" does not necessarily apply to the coping, although in case it does not apply it is better to modify the note to read "Connection angles same as other end." One note at each end for the size of beam to which a beam is coped is sufficient whether one or two flanges are to be cut; in case one end is noted "Same as other end" the size need not be repeated if the same, but it should be given if it is different or if one end is not referred to the other. In general, the note "Same as other end" should be used only to save considerable duplication (page 86 : 4); it should not be used when dimensions or notes can be repeated with no more work. It is sometimes necessary to "block out" the flange of a beam according to special dimensions, as shown in *B* 7, and *H* 14, Fig. 87 or *R* 21, Fig. 93. It is difficult to block out one side of the flange flush with the web because the curved fillet pushes the beam away from the cutter sufficiently to leave a slight projection; the drawing should specify whether or not it is necessary to chip off this projection as in *B* 12, Fig. 87, or *R* 21, Fig. 93.

2. In **office-building construction** great stress is laid upon speed during erection (see (*d*) page 74 : 1), and the beam connections are designed accordingly. Seat angles are used wherever feasible, because the supporting rivets may be driven in the shop, and because each beam may be erected independently of any other beam; this is impossible when two beams are framed to the opposite sides of a web by means of standard connection angles with field rivets in common.

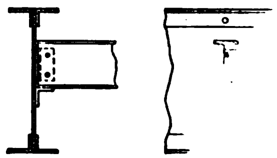


Fig. 89 (*a*).

The type of connection shown in Fig. 89 (*a*) is used for beams supported by other beams or girders which are deep enough to provide space for the seat angles; the whole loads are carried by the seat angles, designed according to page 235 : 1, and the webs are bolted to the side angles to hold the beams in place; ordinarily no holes are provided in the seat angles. Such connections with and without stiffening angles, are shown in Fig. 104, and *B* 12, Fig. 87, is a typical beam supported in this way. The type of connection shown in Fig. 89 (*b*) is used for beams which connect to columns; the seat is designed to carry the whole load (page 232 : 2), and a top angle is used simply to hold the beam in position and to stiffen the connection; the beam is held by two bolts in the top angle and two in the seat angle unless more are required for wind bracing. Such connections, with and without stiffening angles, are shown in Fig. 133, and

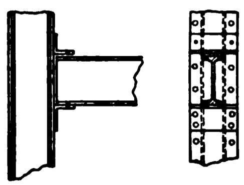


Fig. 89 (*b*).

E 1, Fig. 87, is a typical channel supported in this way. The beams for these types of connection should be ordered with provision for the usual overrun (page 88 : 1) and for easy erection, but they should not be ordered so short that the whole loads rest on the unsupported out-standing legs of the seat angles; unless stiffening angles are used the ends of the beams should extend at least over the fillets of the seat angles and preferably over part of the vertical legs. Beams should be ordered to the nearest half inch in length.

1. **Purlins.** — The beams in the flat roofs of office buildings are usually similar to the corresponding floor beams. The beams may have to slope to give the roof the desired pitch, but often the beams are made horizontal and a slight pitch is provided by varying the thickness at the beams of the concrete or other material in the roof. Steep roof construction such as used in mill buildings is quite different. The roofing is supported by longitudinal lines of "purlins" which are connected to angles on the top chords of the roof trusses or rafters. The type and the spacing of the purlins depend upon the style of the roofing to be used (page 114 : 2). Typical connections for different types of purlins are shown with dimensions on page 315; web connections are provided in every case, but the flanges of only the heavier purlins are connected; extra holes in the webs are used when the purlins act also as struts (page 119 : 1). Channel purlins are used most commonly, usually with the flanges facing up the slope. Purlins are usually made to extend over two bays with "broken joints" in order to stiffen the structure, i.e., the joints in one line are arranged to come at different trusses from the joints in adjacent lines, as shown in the diagram on page 156. The purlins are usually ordered 1″ shorter than the distance between the centers of the trusses, thus leaving 1″ clearance between the ends. For typical purlin details see Fig. 90.

2. Holes should be provided in the webs of beams for **tie rods** when rods are necessary to resist the thrust of floor arches. The number and the size of the rods are usually determined by the designing department; * the most common sizes are $\frac{5}{8}$″ and $\frac{3}{4}$″ in diameter. The holes are usually made the same size as others in the web for con-

* For the method of design, see page 201 : 3.

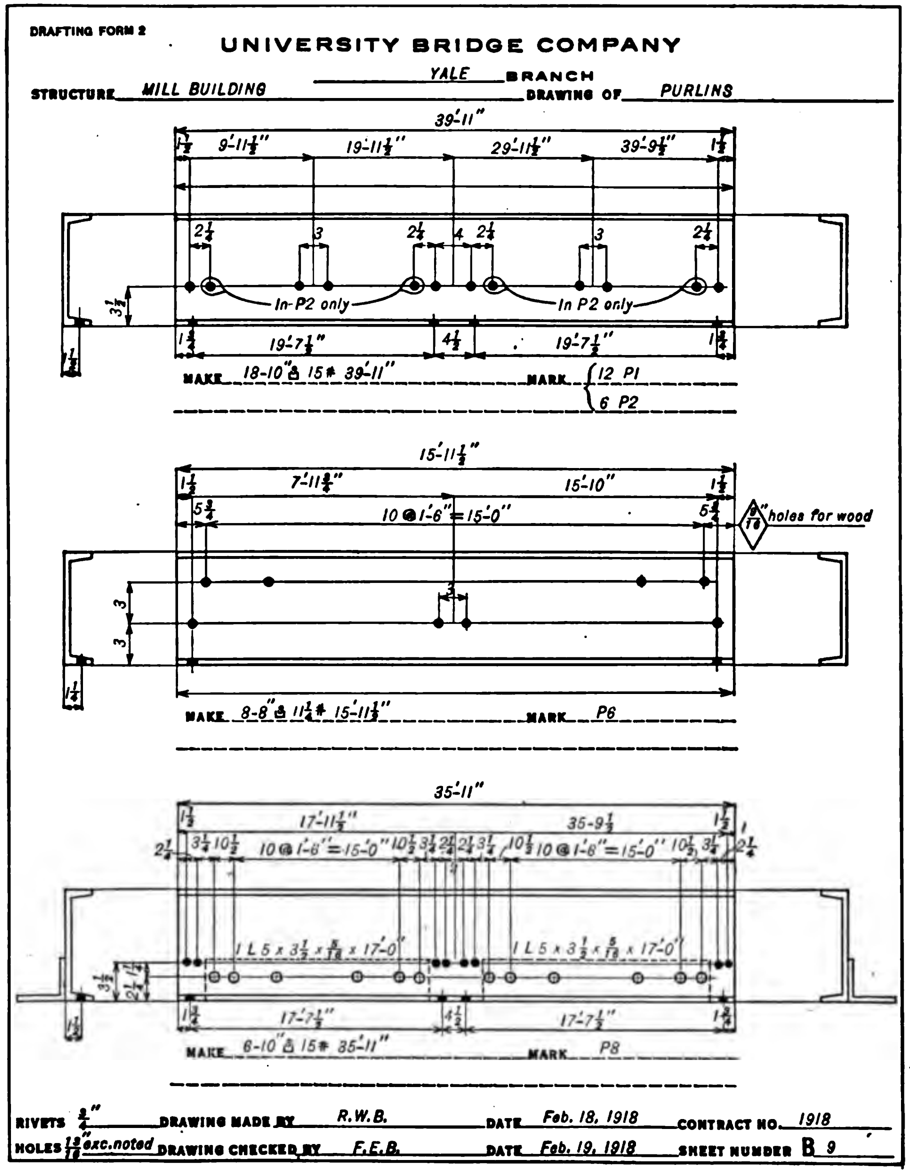


Fig. 90. Typical Purlins.

venience in punching them; they are made in pairs (3″ apart) to provide for the rods in adjacent panels as indicated in the floor plan in Fig. 158. The location of the holes vertically depends upon the floor construction because the rods should be placed where they will best serve their purpose; this depends upon the type of arch, and upon whether the arch is supported by the top or bottom flange or by a skewback angle riveted to the web (Fig. 87). On account of the use of different depths of beams in the same floor, the elevation of the rods is usually determined at the outset and noted on the plans, as shown in Fig. 158.

1. Holes should be provided in the webs of purlins for **sag rods.** The function of the sag rods is to give intermediate support to the purlins at right angles to their webs; this is necessary because the resultant forces for which the purlins are designed are not parallel to the webs. One line of rods is used if the span between trusses is from 15 to 20 feet and two lines if more than 20 feet. The upper purlins should be tied to each other, or to a strut at the ridge by means of bent rods (Fig.175 (*b*). Rods from $\frac{3}{8}''$ to $\frac{5}{8}''$ in diameter are used but $\frac{5}{8}''$ is the most common size. The holes are usually made the same size as others in the web provided they are small enough to leave sufficient bearing for the nuts. The holes are made in pairs (3″ apart), either in the center of the web or on line with the holes for the truss connections (see next paragraph).

2. When a **multiple beam punch** is to be used all the rivets and holes in the web of an I-beam or a channel should, if possible, be so located that the beams will not have to be shifted laterally as they pass through the punch. Holes for tie rods, sag rods, and anchors may be moved slightly to meet this requirement.

3. Wooden floors and wooden sheathing are attached to steel beams or purlins by means of wooden **nailing strips or spiking pieces** which are bolted to the steel. The nailing strips are usually bolted to the webs of channel purlins and to the top flanges of beams, with $\frac{1}{2}''$ bolts. $\frac{9}{16}''$ holes must be provided for these bolts at intervals of from 1′–0″ to 1′–6″, preferably in multiples of 3″ to permit the use of standard strip templets; in the flanges of I-beams the holes should be staggered. It is well to note that these holes are "for wood" so that shopmen need not waste time in useless refinements. See *B* 16, Fig. 92, and *P* 6, Fig. 90. Care should be taken to space the holes in the flanges of beams far enough from the ends to allow for any underrun or any coping, and far enough from the connections for any other beams which might prevent the insertion of the bolts.

4. **Holes in the flanges** of I-beams and channels may sometimes be dimensioned in the front and the end views in order to dispense with the top and bottom views, as in Figs. 90 and 92. When there is a large number of holes, or when the drawing might be misinterpreted, the flange views should be shown; for example, staggered holes should regularly be shown, as in *B* 16, Fig. 92.

5. **Beam Girders.** — In order to give greater strength or greater bearing, two I-beams or an I-beam and a channel may be used side by side. To distribute the loads and make them act as a single member they should be bolted together with separators of some form between them. Cast-iron separators are commonly used, but for heavy work, or for beams of unequal depth, special diaphragms made of I-beams, or of plates and angles, are used. For lighter use, gas-pipe separators may serve, particularly if they are required simply for holding the beams at fixed distances apart. Gas-pipe separators are used between grillage beams with a single rod passing through all the beams of one tier. (See Fig. 291). Gas-pipe and cast-iron separators are shown on page 316. They are not drawn in detail, but are simply listed on combination shop and shipping bills (page 174 : 1). The separators are spaced about four or five feet apart, and from six inches to one foot from the ends. If the beams have rigid connections at the ends, the end separators may be omitted. The I-beams and the separators are usually shipped separately and assembled in the field (*C* 17, Fig. 92), but this depends upon the number and the size of the beams, and upon the practice of the individual companies. If they are assembled in the shop the two beams should be detailed together and called a girder, as *G* 19, Fig. 92; a list of separators and bolts should then be given on the drawing and on the corresponding shop bill; on the latter, reference should be made to the shop and shipping bills from which the separators and bolts are made and these bills should bear the note: "Send to the shop for assembling."

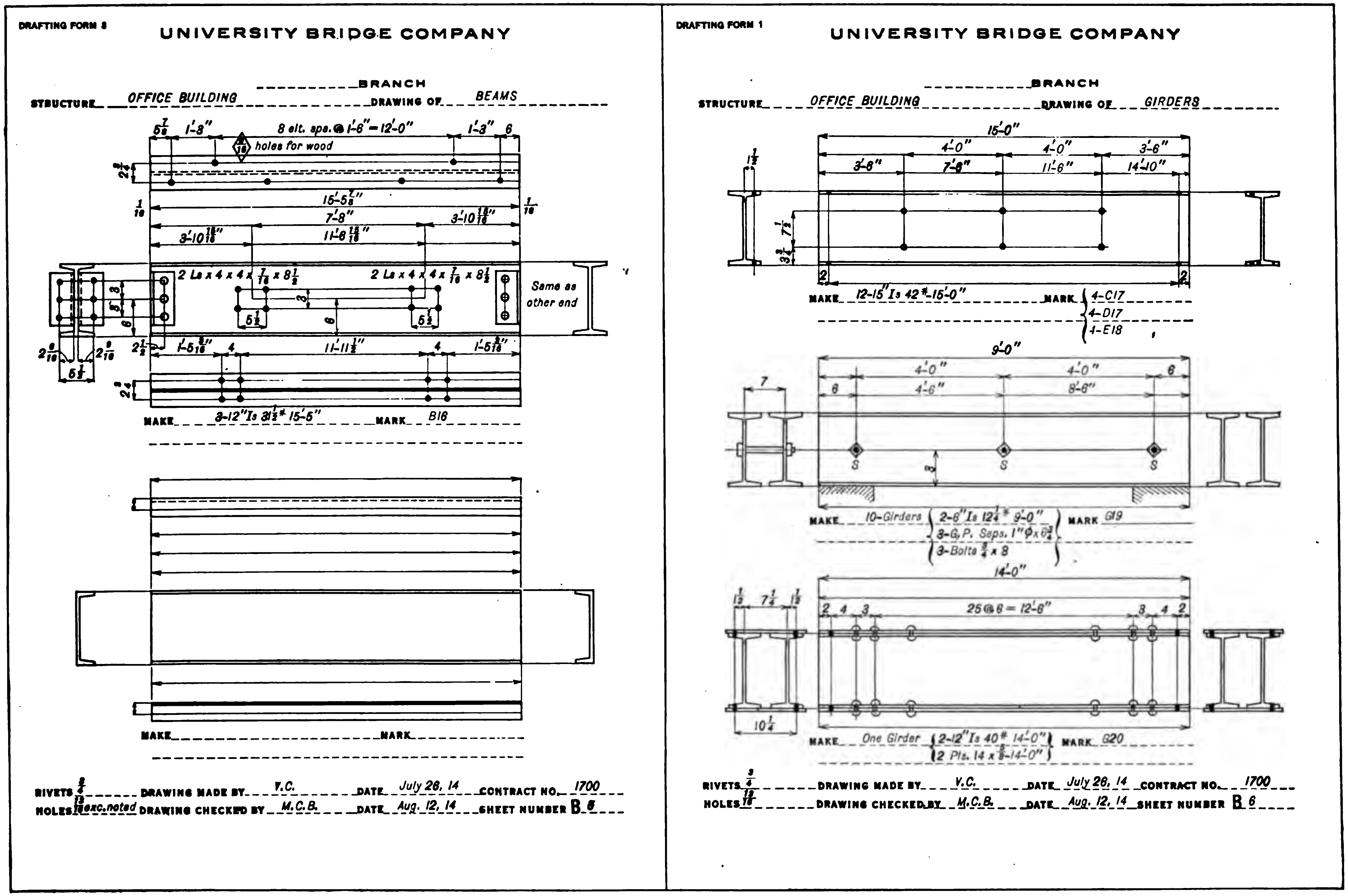
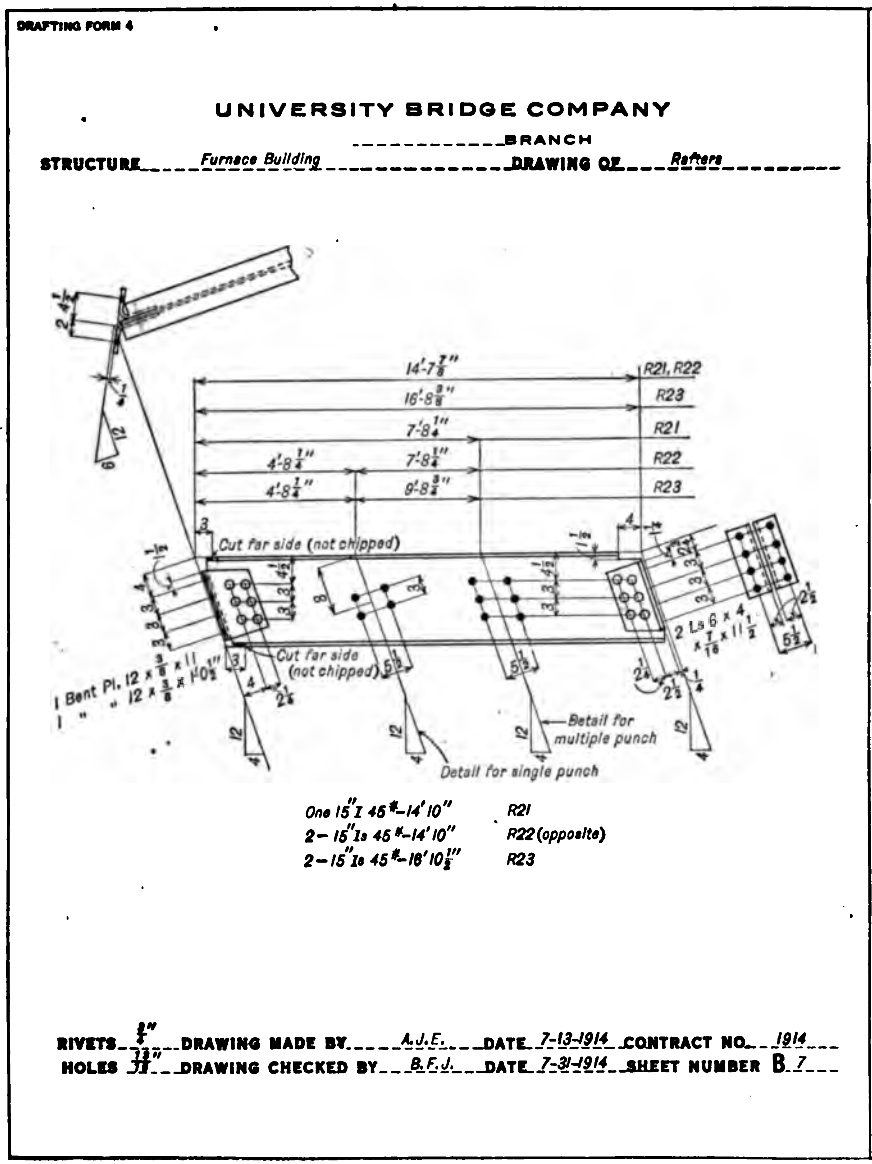


Fig. 92. Typical Beams.



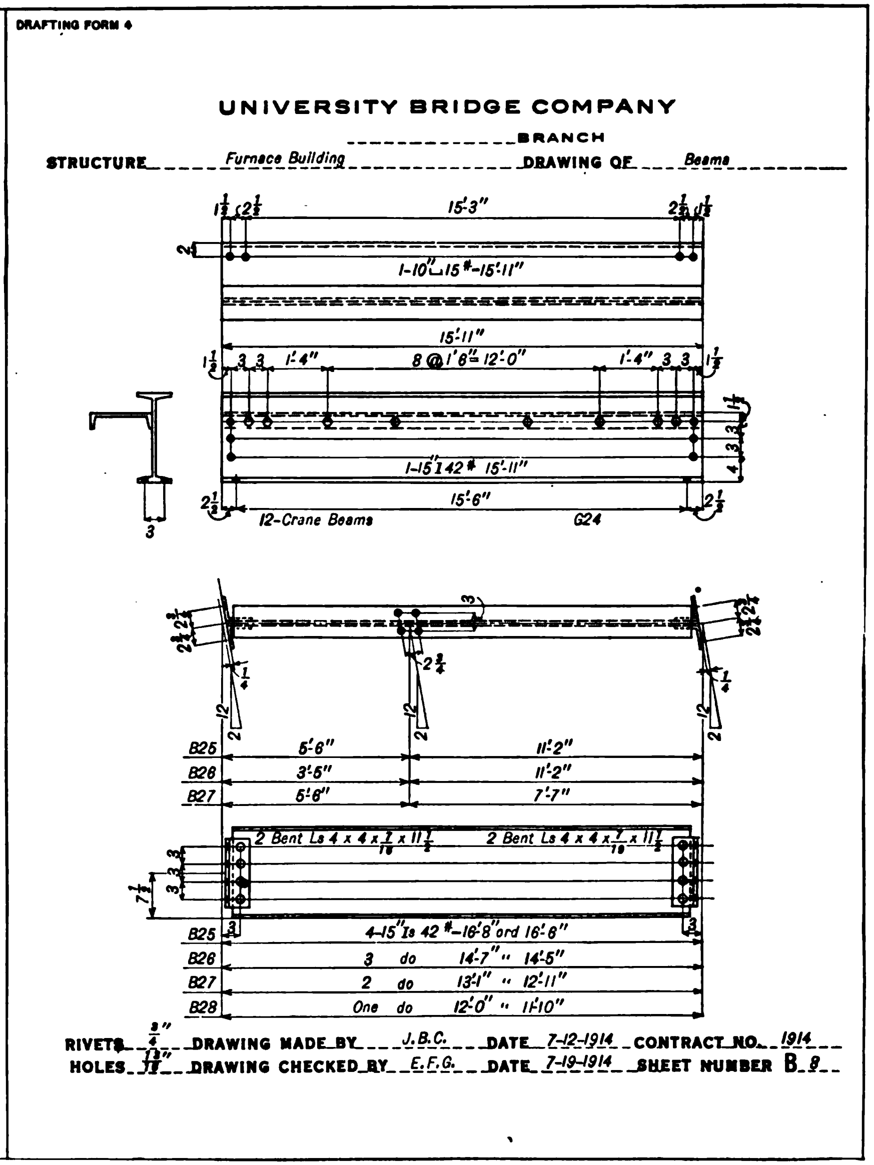


Fig. 93. Typical Beams.

Special forms are often printed for double I-beam girders, but the form for single I-beams may be used by tracing an additional I-beam or channel in the end views from another sheet, as in *G* 19 and *G* 20, Fig. 92.

1. When I-beam girders with separators are insufficient, plates may be added to make a **box sec ion,** the separators being omitted as in *G* 20, Fig. 92. The rivets are spaced 6″ apart, with one or two smaller spaces near the ends. Obviously, rivets through the inner flanges would be inaccessible.

2. **Wall plates** or flange plates are sometimes riveted to the top flanges of beams which support masonry walls in order to furnish wider bearing. These plates are not necessarily concentric with the beams.

3. **Skewback angles** are sometimes riveted to the webs of beams to provide supports for floor arches at the proper elevations. Rivets should be spaced about 1′-0″ apart, as shown in *B* 10 and *H* 14, Fig. 87. Similarly, stiffening angles are often riveted to purlins which act as struts, as shown in *P* 8, Fig. 90, and *S* 2, Fig. 147. These rivets may be spaced 1′-6″ apart.

4. **Crane runway beams** are sometimes stiffened laterally by channels riveted as shown in *G* 24, Fig. 93. The rivets are placed at intervals of about 1′-6″, except at the ends where about two 3″ spaces are used. For export work these channels and the I-beams should be shipped separately.

Fig. 94.

5. For **skew connections** see page 146 : 5. Note that it is usually better to cope out one flange of a beam to give desired clearance, as in Fig. 93, than to require a diagonal cut which must be sawed. Even though the web must be cut diagonally, the flanges may often be blocked out to avoid sawing, as in Fig. 94.

CHAPTER XVIII

PLATE GIRDERS

SYNOPSIS: Specific suggestions for drawing typical plate girders.

1. Plate girders are used extensively in every form of steel construc-on, because of their **adaptability**. They resist transverse bending like eams (page 83 : 1), but they are used for heavier loads, for longer pans, or for conditions which the single rolled beams do not satisfy. With different depths, different forms of flanges, or different sizes of component parts, girders may be made to serve a great variety of purposes.

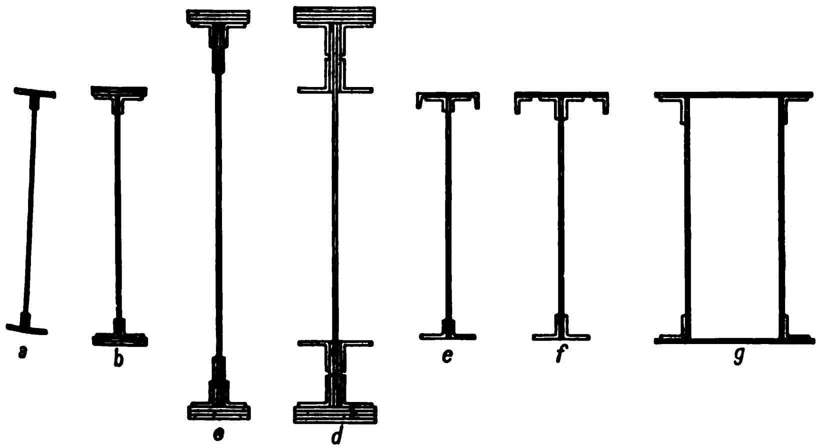


Fig. 95. Typical Girder Cross Sections.

2. **Types.** — Girders may be composed of one or more web plates, with nple or composite flanges, as illustrated in Fig. 95. The most com-on type of cross section, shown at (*a*) and (*b*), is composed of a single eb plate and four angles, with or without one or more cover plates on each flange. This form may be adapted to suit most requirements, the heavier types of flanges being used only in special cases. Only the more common girders are illustrated in this chapter.

3. **Main Dimensions.** — The length of a plate girder which is dimensioned on the drawing is usually the extreme length out to out; when some other dimension is seemingly more important it may be given instead, as for example the distance from center to center of end holes when the end connection is of a type similar to the alternate form shown in the corner of Fig. 99. For convenience in the drafting room the distance from center to center of supports is usually given also, as in Figs. 98, 100, or 103. The distance from the end of the girder to the center of the support should provide for ample clearance, as explained on page 73 : 1. The *nominal* depth of a girder is usually the depth (width) of the web plate, for this is generally in even inches and often in even feet or half feet; the depth *dimensioned* upon the drawing is invariably the distance from back to back of flange angles, which is $\frac{1}{2}''$ (or $\frac{1}{4}''$) more than the depth of the web plate. The flange angles thus project beyond the edges of the plate to allow for any irregularities in the latter which may result from rapid shearing in the mill (page 25 : 3); if part of the plate should project beyond the angles it would have to be chipped off before the cover plates or other parts could be put in proper position against the angles. Unless the upper edge of the plate is exposed to the weather, the depth of the girder from back to back of flange angles is usually made $\frac{1}{2}''$ greater than the depth of the web plate, allowing $\frac{1}{4}''$ variation on each edge. In bridge and viaduct work and in other structures in which the girders are exposed to the elements, the upper

edges of the web plates are made flush with the backs of the angles unless cover plates are used; otherwise, a rain pocket is formed which will lead to a more rapid deterioration of the girder. The distance back to back of angles is thus $\frac{1}{4}''$ greater than the depth of the web plate for exposed girders without cover plates. It is well to add a note stating whether or not any projections which may occur should be chipped off. See Figs. 98 and 99. It is not feasible to draw extra lines to represent the edges of the web plate, the only indication of the difference in depth being the discrepancy between the billed width of the plate (usually in even inches, page 43 : 3) and the dimension back to back of angles (usually with a fraction).

1. Since the vertical shearing stresses of a plate girder are resisted by the web plate, they must be transmitted from the web to the supports. Sometimes the web plate is riveted directly to the face of a column or to the stiffening angles of another girder (Fig. 99); but more often **end stiffeners** are used either to serve as connection angles (Figs. 98, 100 or 104), or to transmit the stresses to the masonry or to the column seat upon which the girder rests (Figs. 101, 102, or 105). The size of the end stiffeners and the number of rivets in them are determined as explained in Chapter XXXIX, page 266. Stiffening angles should be made to bear against the outstanding legs of the flange angles; since they are placed in contact with the vertical legs of the flange angles, they must be cut to fit the curved fillets, as shown in Fig. 267 (*b*). The billed length of the stiffeners is the exact distance in the clear between the outstanding legs of the flange angles; the ordered length is made $\frac{1}{4}''$ greater (see next paragraph). For suggestions regarding the spacing of the rivets see page 70 : 4 (*b*). The spacing should ordinarily be made symmetrical about the center line, so that the stiffeners on opposite sides of the web are interchangeable; but if the holes for a connection to the outstanding legs are necessarily slightly unsymmetrical, it may be deemed advisable to space at least one rivet through the web unsymmetrically to prevent the possibility of the stiffeners being assembled upside down.

2. The **flange angles** usually extend the full length of a girder and should be billed accordingly. They are ordered about $\frac{3}{4}''$ long and are then cut to the required length in the shop where they can be cut with greater precision than at the mill (page 28 : 1). The stringers and floor beams of bridges are often milled at the ends; otherwise the angles are sheared. The extra $\frac{3}{4}''$ is indicated on the material order bills and on the shop bills (pages 165 : 1 and 167 : 4). **Web plates** should extend to the extreme ends of a girder when they are to be milled or when no stiffeners are placed with their outstanding legs at the extreme ends of the girder; when stiffening angles are so placed the web plates may be billed long enough to come within $\frac{1}{4}''$ or $\frac{3}{8}''$ of each end. In the larger girders the web plates must be spliced because it is impossible to obtain plates long enough to extend the full length of the girder. For the location and the design of web splices see page 270 : 4. The lengths of the web sections should be billed to allow from $\frac{1}{4}''$ to $\frac{3}{4}''$ between them at the splices (compare page 165 : 2).

3. Unless the web plate of a girder is thick enough to resist the shearing stresses, it must be reinforced by **intermediate stiffening angles,** as explained on page 266 : 2. When the position of stiffening angles is definitely determined by members which are to connect to the outstanding legs, the stiffeners must be spaced before the flange rivets; otherwise it is better to fix the flange rivet spacing first, and then place the stiffeners at those rivets which are located most suitably to give the best spacing (page 269 : 3). It is customary to place the stiffeners so that the backs of the angles are toward the *nearer* end of a girder. The rivets in the intermediate stiffeners should line up with those in the end stiffeners even when a smaller number is used; this saves extra dimensioning, and it simplifies the shop work, particularly when multiple punches are used (page 29 : 5). Instead of using the full number of rivets used in the end stiffeners, some may be omitted unless the resulting spaces exceed the allowed maximum (page 69 : 1), or unless the full number of rivets is needed for other reasons (as for example, when the stiffeners serve as connection angles for other members or when they are placed at web splices).

4. Stiffening angles overlap the vertical legs of the flange angles, and unless they are crimped (see next paragraph) spaces are left between the stiffeners and the web as shown in Fig. 97. Plates called "**fillers**" are inserted to "fill" these spaces so that the rivets can be effectively driven without bending the angles out of line, and so that no surfaces will be left inaccessible for painting. The width of a filler should be the same as

the width of the superimposed stiffening angle unless the filler is made to extend under two or more angles, or made wide enough to take an additional row of rivets according to page 235 : 2. The thickness of the filler should be the same as the thickness of the flange angles, unless part of this space is occupied by a splice plate or a reinforcing plate. In this case the filler should be thick enough to make up the difference; fillers less than $\frac{3}{16}$″ are not used, and smaller differences should be made up by making the thickness of the splice plates or reinforcing plates equal to that of the flange angles. If the thicknesses of the top and bottom flange angles differ by $\frac{1}{16}$″ the filler may be made of either thickness. If they differ by $\frac{1}{8}$″ the filler may usually be made the mean thickness. If they differ by more than $\frac{1}{8}$″ two fillers should be used, one as thick as the thinner angle and the other equal to the difference in thickness; this second filler should extend to the fillet of the thinner angle. The length of the filler should preferably be made about $\frac{1}{2}$″ less than the clear distance between the flange angles, allowing the usual shop clearance of $\frac{1}{4}$″ at each end. On girders which are exposed to the weather it is well to reduce this clearance one-half to leave less chance for water to enter, but due allowance should be made for the overrun of heavy angles (page 25 : 1). Some specifications require the fillers under end stiffeners to fit tightly at the bottom.

Fig. 97.

1. Intermediate stiffening angles are sometimes "crimped" or bent so that they are brought into contact with the web, as shown in Section BB, Fig. 102. No fillers are required under **crimped stiffeners.** Stiffeners which transmit direct stress should not be crimped because straight angles are more effective; thus end stiffeners or stiffeners under concentrated loads should not be crimped. Similarly stiffeners which have holes in the outstanding legs for the connection of other members should always rest upon fillers, because better results can be obtained in this way. Most specifications permit the crimping of all other intermediate stiffeners although not all companies are well equipped for this work. Many companies prefer to furnish fillers, particularly when the cost of the additional material is met by the customer, as in contracts based upon a price per pound. The billed length of a crimped stiffening angle should include the amount of metal required for each crimp, which is equal to the depth of the crimp, i.e., the thickness of the flange angle. Thus the length of a crimped stiffener of a girder is equivalent to the depth of the girder from back to back of flange angles. An extra $\frac{1}{2}$″ should be ordered for each crimp, so that the angles can be cut to fit properly after they are bent.

2. **Cover plates** may be used on plate girders to furnish additional metal in the flanges. Since the flange stress is a function of the bending moment, the greatest flange area is required where the moment is maximum; as the moment decreases, the flange area may be reduced. This reduction is effected by cutting off the cover plates successively at points beyond which they are no longer needed, as explained in Chapter XXXVIII, page 259. If a girder is to be exposed to the weather, one plate on the top flange and sometimes one on the bottom flange are made to extend the full length of the girder in order to protect the surfaces of contact between the angles and the web from the action of the elements. Similarly, all the cover plates on the top flanges of crane runway girders must be continuous in order to furnish uniform bearing for the rails which rest directly upon them. The cover plates on the top flange of a railway deck girder need not necessarily be made full length, since the ties may be "dapped" (i.e. notched), different amounts to make up for the differences in plate thickness; the ties must be notched, also, for the rivet heads. Special detailed drawings are often prepared in the drafting room for the ties of a bridge, especially when they have to be sawed to provide for the super-elevation of the outer rail on a curve. The cover plates may be billed with the flange angles (Fig. 105), or they may be billed on special dimension lines with the distance from the end of each plate to the end of the girder, as shown in Fig. 101. To save space, all the plates of one flange may be billed as in Fig. 102, provided portions of the line are omitted so that the overlapping dimensions may be distinguished more easily. Universal Mill plates with rolled edges are usually ordered for cover plates, particularly for girders which are exposed to the weather (page 25 : 3). Full length plates are ordered $\frac{3}{4}$″ long the same as the flange angles (page 96 : 2); other plates are ordered the same as the billed lengths.

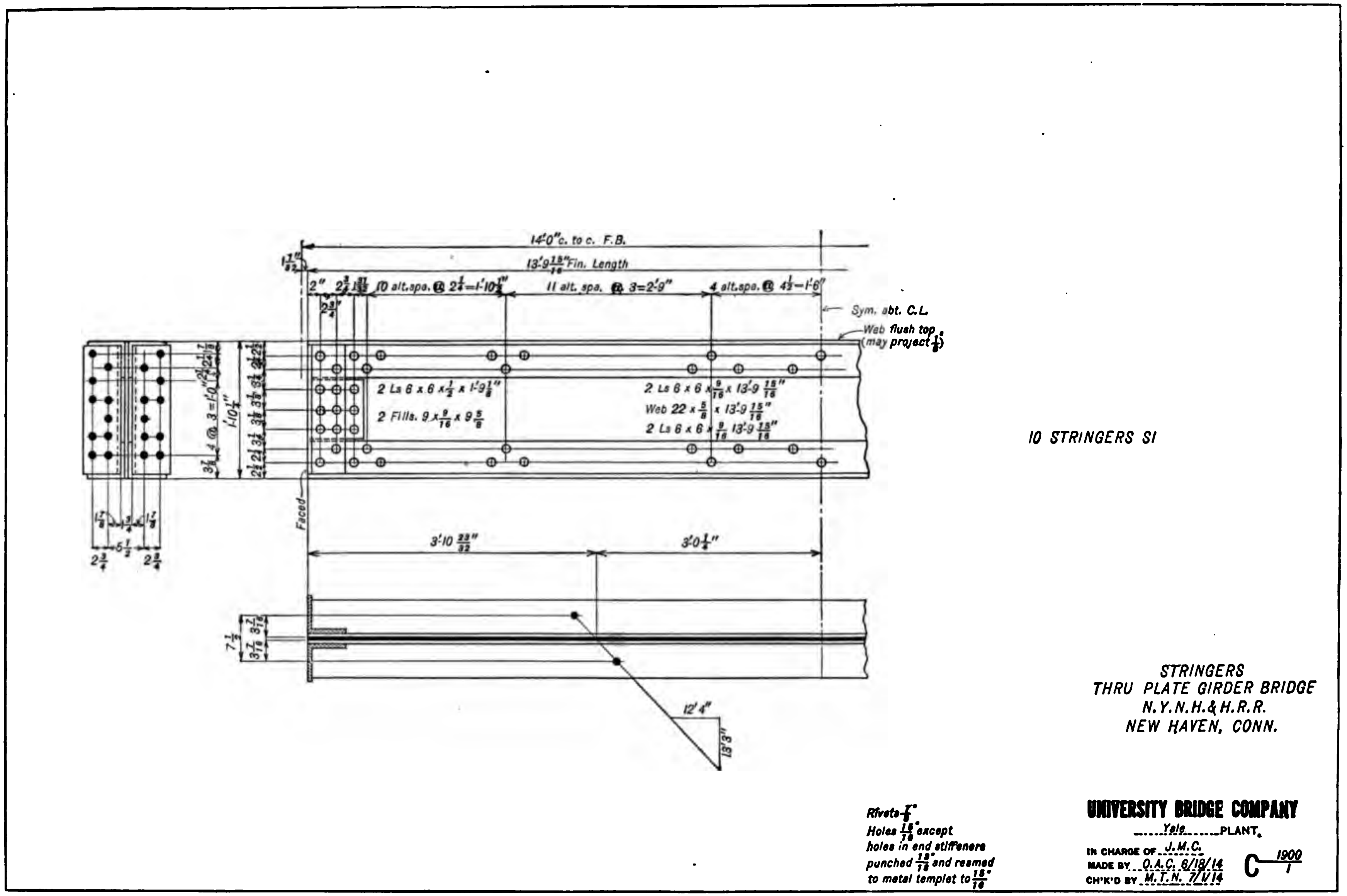


Fig. 98. Railroad Bridge Stringer.

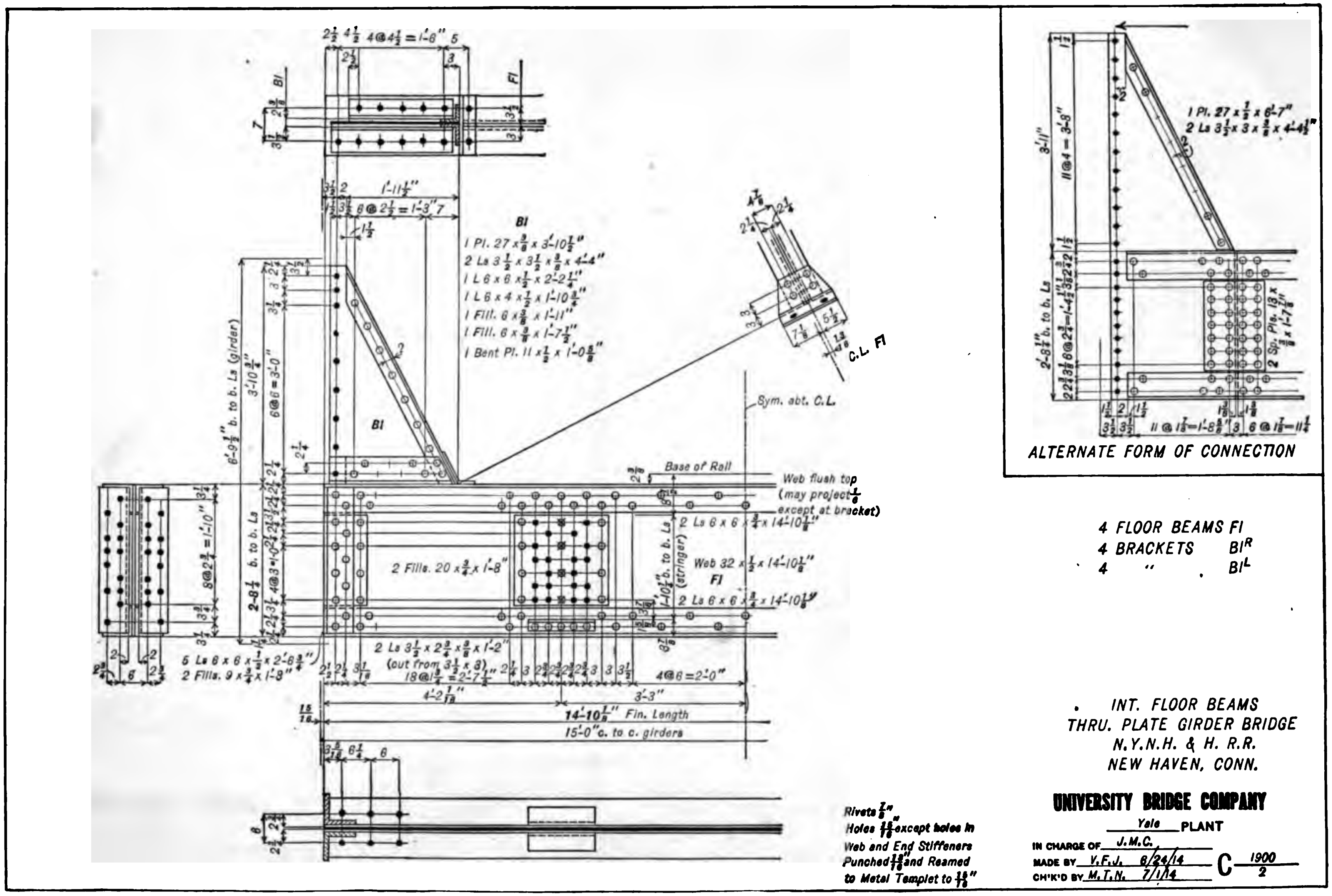


Fig. 99. Floor Beams for Through Railroad Girder Bridge.

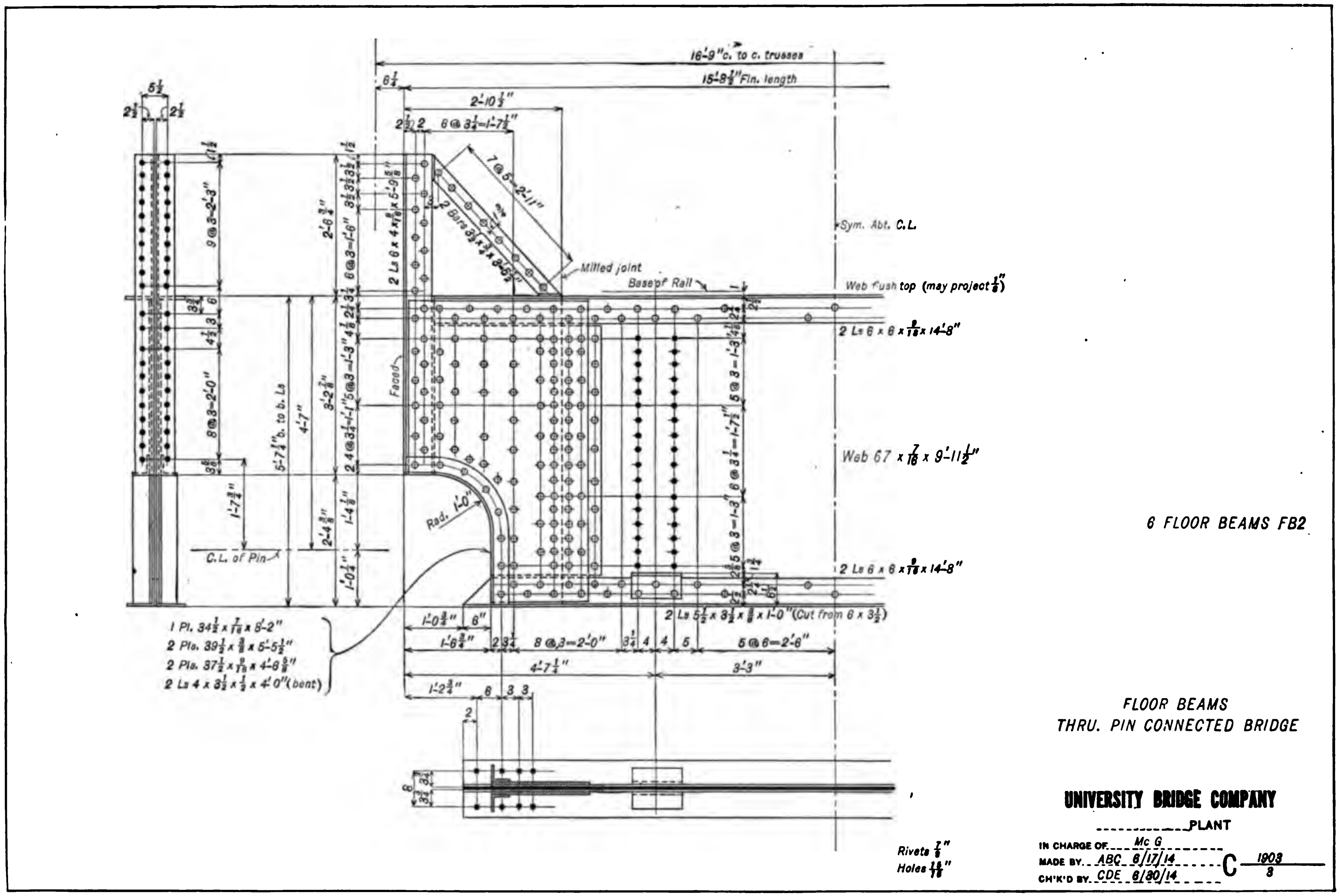


Fig. 100. Floor Beam for Pin-connected Truss Bridge.

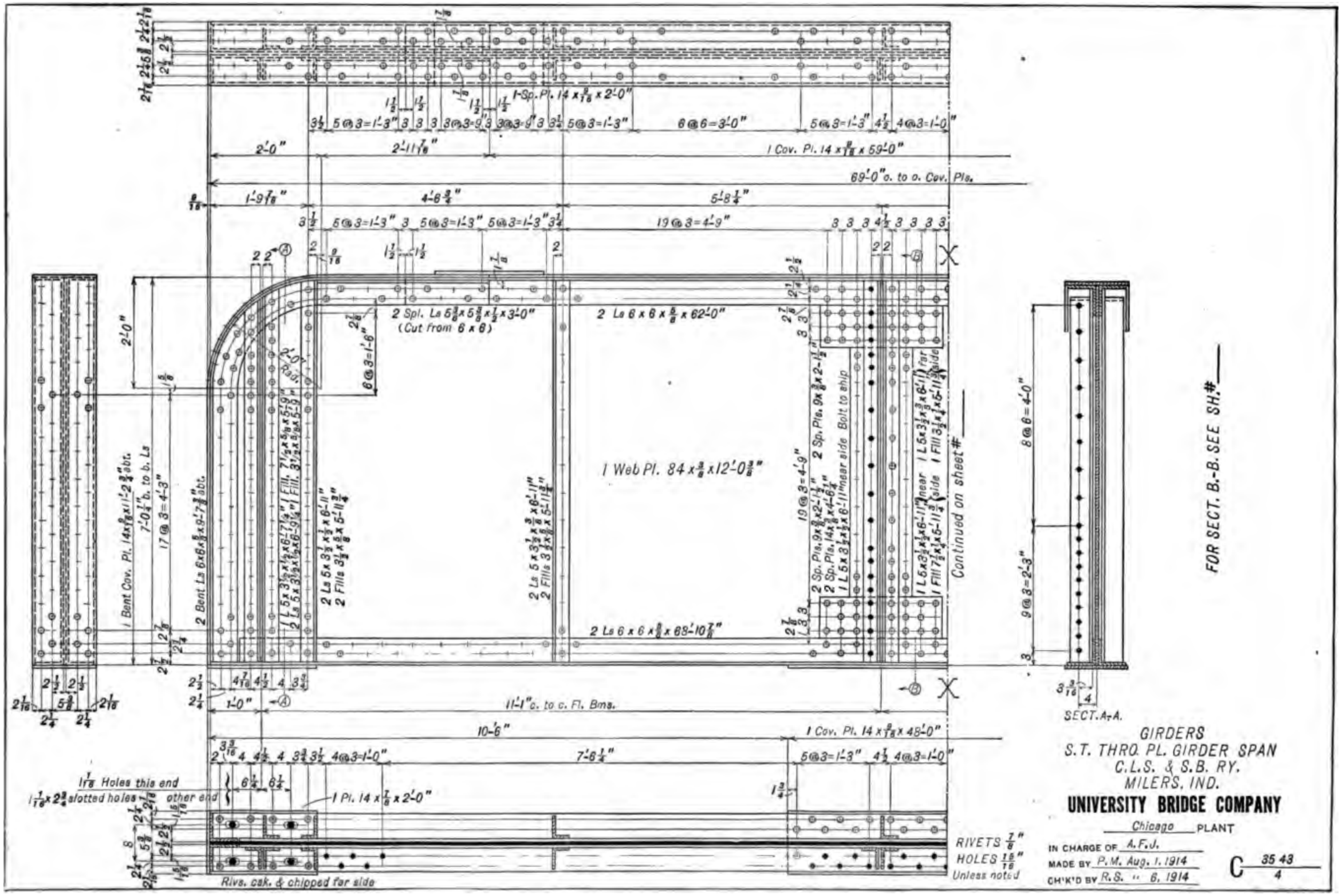


Fig. 101. Through Railroad Bridge Girder.

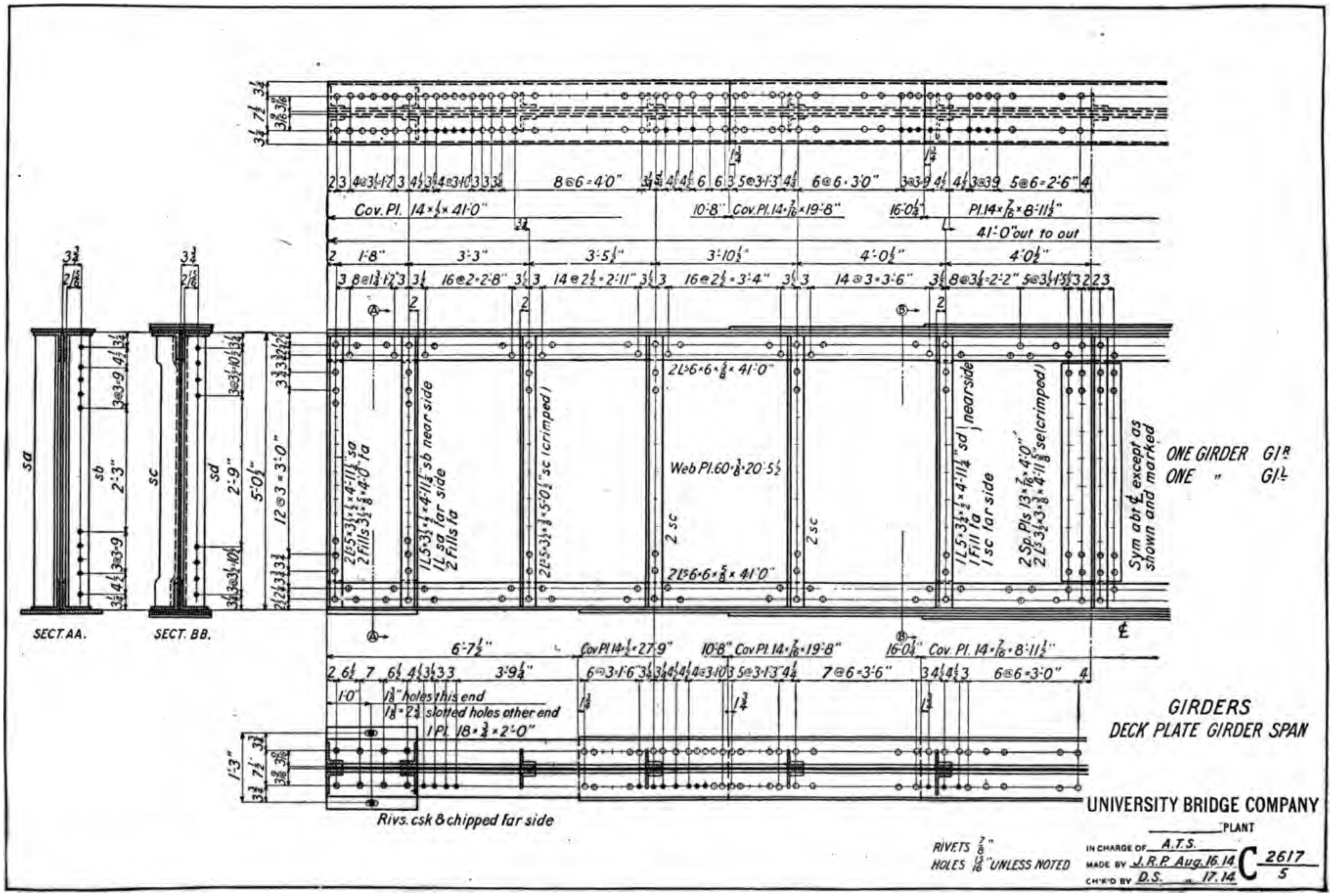


Fig. 102. Deck Railroad Bridge Girder.

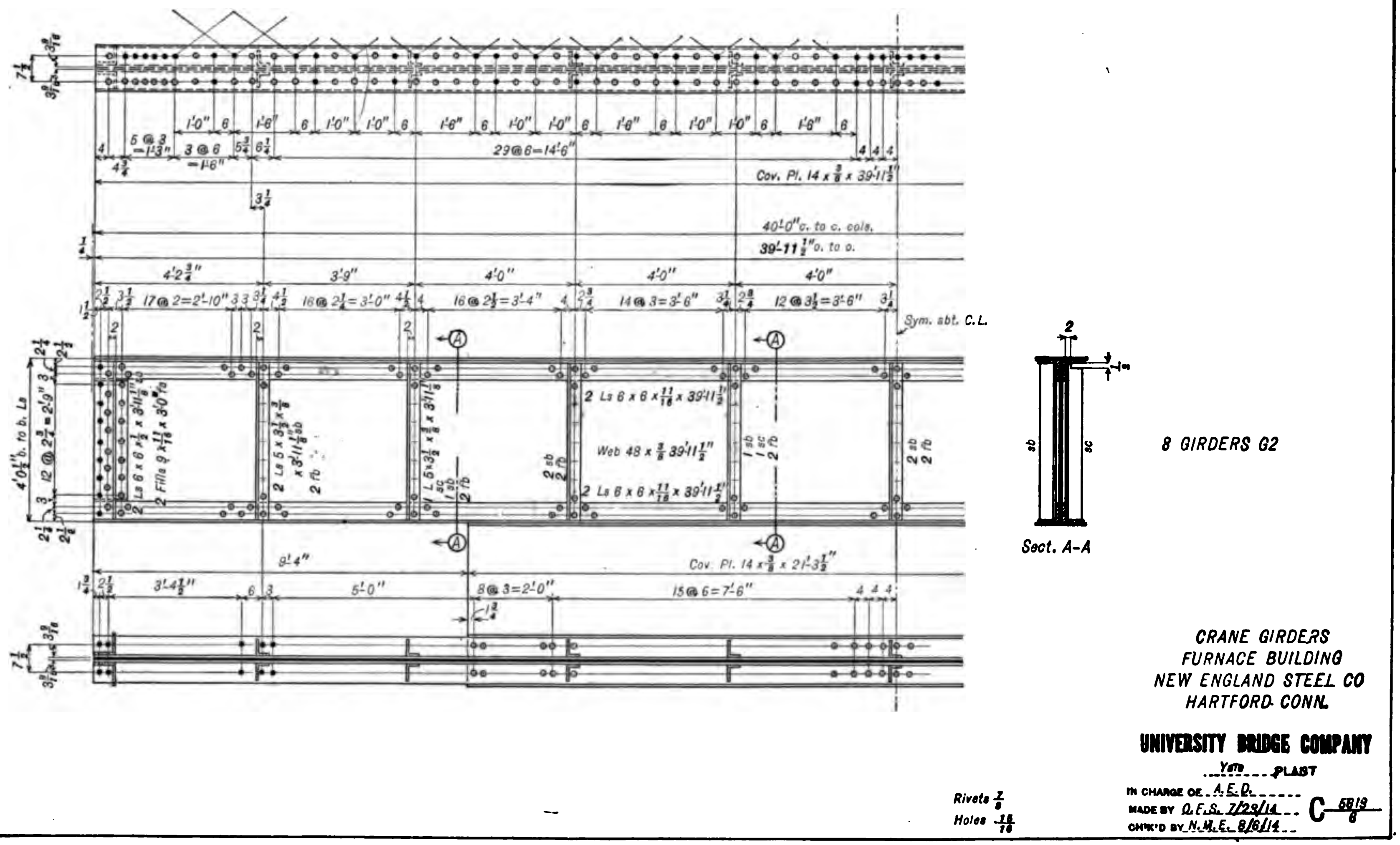


Fig. 103. Crane Runway Girder.

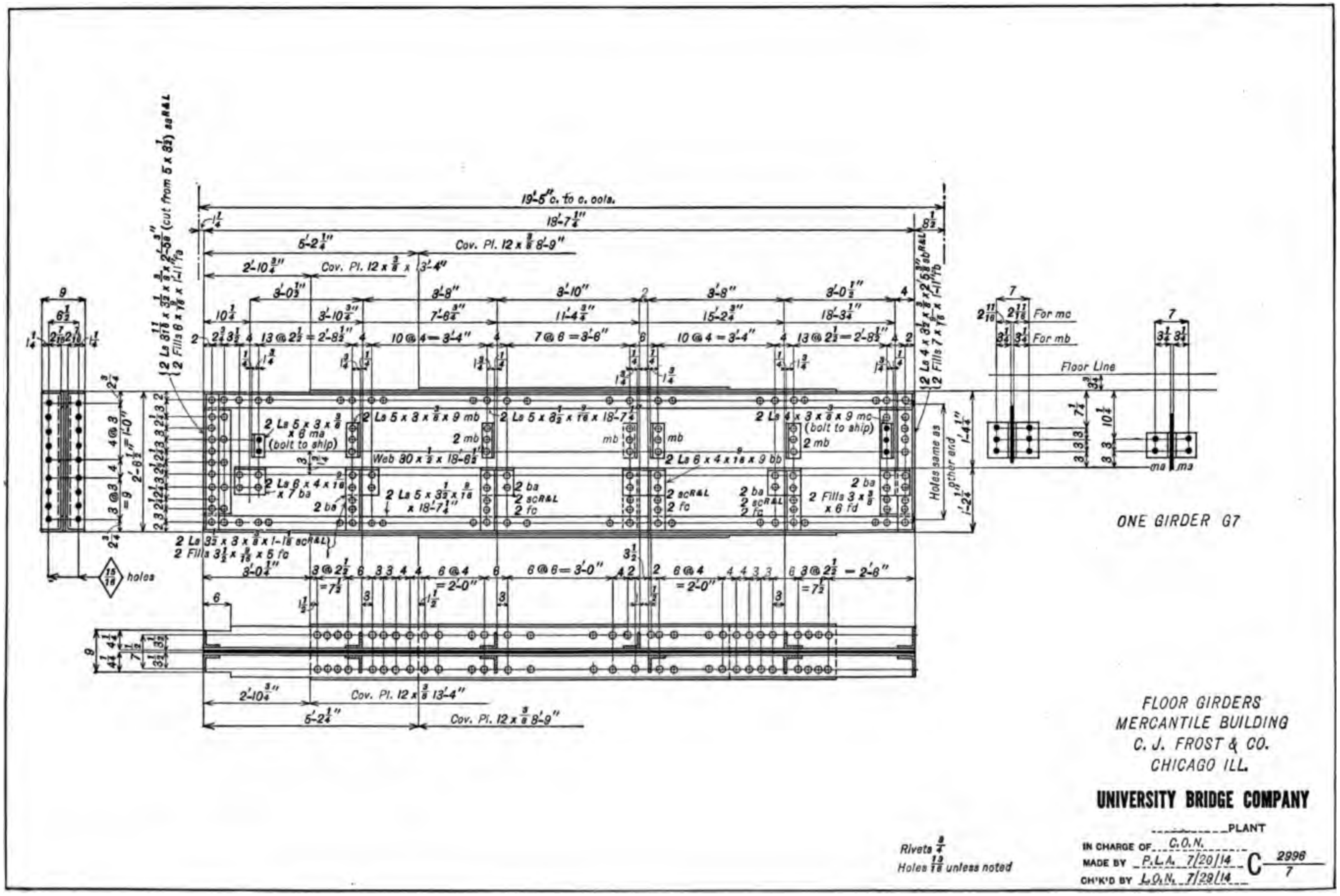


Fig. 104. Floor Girder.

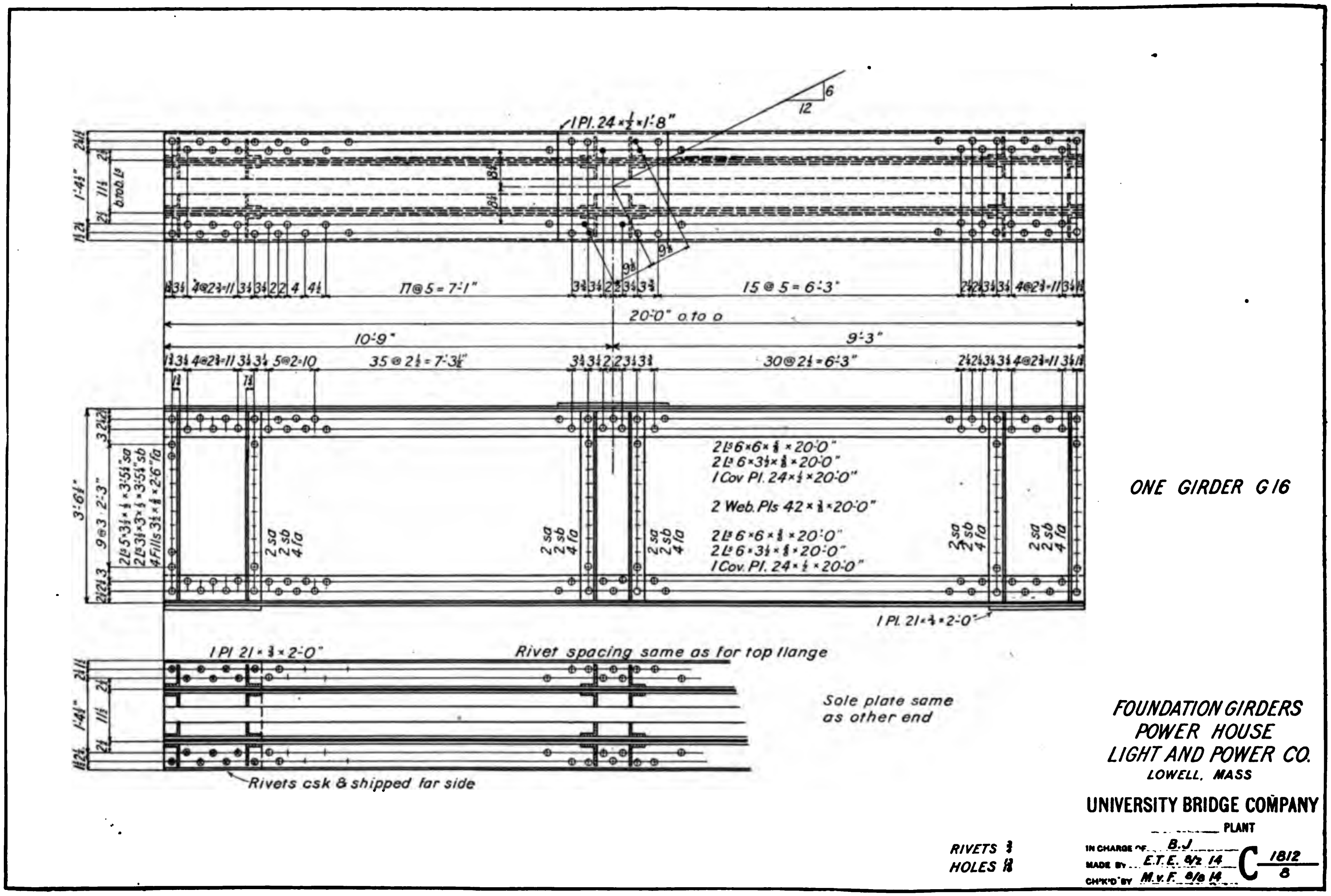


Fig. 105. Box Girder.

1. **Flange Rivets.** — The flange angles of a girder are fastened to the web plate by sufficient rivets to transmit the flange stress for which the angles are designed. The rivets are usually closer together toward the ends of a girder than they are near the center. The pitch for each section of a girder must be determined for the conditions of loading and the proportions which are peculiar to that girder. The maximum pitch for each panel is found as explained under the proper case in Chapter XXXVII, page 241. The minimum pitch depends upon the strength of the web, as explained on page 255 : 2. Between these limits the draftsman should space the rivets according to the general rules for rivet spacing given in Chapter XIII, particularly pages 69 : 1, 70 : 2, and 70 : 4; the spacing of the rivets should be made the same in both flanges for the benefit of the draftsman and the shopmen.

2. **Rivets in Cover Plates.** — The rivets which fasten the cover plates to the flange angles must satisfy the conditions given on page 263 : 3, but except in heavy or unusual work, the spacing is governed by the general rules for rivet spacing given in Chapter XIII, page 68. A single line of rivets is often used in each 5″ or 6″ angle even if a double line is used in the vertical leg. Extra rivet lines are placed in wide cover plates, as explained on page 69 : 2. Holes for lateral bracing should be spaced before the stiffening angles are located for usually the angles can be so placed that their outstanding legs will not interfere with the holes which best meet the requirements of the lateral plates. The flange rivets and the stiffening angles should usually be spaced before the remaining rivets in the cover plates. No rivets in the cover plates should be placed so near the outstanding legs of the stiffening angles that they cannot be driven by machine (page 73 : 5); it is well, therefore, to tie the nearest rivet through the cover plate to the rivet line of each pair of stiffeners by a line or a dimension, as in Fig. 102, in order to show that this point has not been overlooked. The rivets in the plates should be so placed that the same templets can be used for the top plates and angles that are used for the bottom plates and angles; additional rivets or groups of rivets may be spaced differently to accommodate connecting members, but the remaining rivets should be opposite (see (*d*), page 70 : 2). The rivets near the end of each cover plate should be spaced not over four diameters as explained in (*d*), page 69 : 1; this applies to plates on the tension flanges as well as those on the compression flange so that the strength of the plates may be developed within a comparatively short distance. Since all cover plates which do not extend full length of the girder should be used as ordered, the end rivets should be placed $1\frac{3}{4}''$ from the ends of the plates to provide ample edge distance in case the plates underrun (page 165 : 2).

3. **Standard gages** are not necessarily used in the outstanding legs of the flange angles or of the stiffening angles. It is usually preferable to change the gage sufficiently to make the distance from center to center of rows a multiple of $\frac{1}{2}''$, and the distance from the center of the web to either row a multiple of $\frac{1}{4}''$; in this way small fractions are used in the gages only, and are avoided in the cover plates and on the drawings of members which connect to the flanges or to the stiffeners. In order to eliminate thirty seconds from the gages, web thicknesses in sixteenths should be considered $\frac{1}{16}''$ greater; this incidentally allows for paint, scale, or bends which might tend to separate the surfaces of contact.

4. When the **outstanding legs** of two stiffening angles are in contact they need not as a rule be riveted together. On girders which are exposed to the weather these outstanding legs should be riveted at intervals of 1′ 0″ if the girders are 3′ 0″ or more in depth. It is preferable where possible to place such stiffeners at least 2″ apart so that they can be painted.

5. **Holes for anchor bolts** should be $\frac{5}{16}''$ or $\frac{3}{8}''$ larger than the bolts (Fig. 102); this facilitates the placing of a girder if the anchor bolts have been set, and it provides for drilling holes in the masonry if the bolts are to be placed after the girder is in position. The holes at one end should be slotted to allow for expansion and also for inaccuracies in setting the bolts. Provision for expansion to the extent of $\frac{1}{8}''$ for each 10′ 0″ should be made in all bridge girders. When cast-iron pedestals are used (see diagram, Fig. 153) the holes in the girder are made only the usual $\frac{1}{16}''$ larger than the bolts, but at one end the holes should be slotted (Fig. 101).

6. **Reference dimensions** are often given on the drawings for use in the drafting room; for example, dimensions to the base of rail (**Fig. 100**), or to the floor line (Fig. 104).

1. When more than one sheet must be used to properly illustrate a long girder a **reference line** should be used on each sheet to indicate the **points** which are common to both drawings, as illustrated in Fig. 101 (compare page 121 : 2).

2. **A box girder** with more than one web is shown in Fig. 105.

3. A girder is sometimes "**cambered,**" i.e., curved in a vertical plane, to prevent the center from sagging lower than the ends. The amount of camber should equal the maximum deflection so that the girder will assume a horizontal position under a full load. The camber is effected partly by the proper rivet spacing, but mostly by careful shop work; in fact, slight cambers may be made entirely by the fitters and riveters by placing the proper supports under the girders while assembling and riveting them. Since a web plate cannot be curved except by elaborate cutting, the usual camber is provided at the web splices by spacing the rows of rivets in the splice plates farther apart at the top than at the bottom so as to separate the ends of the adjacent web sections more at the top. The corresponding spaces in the top flange angles are made greater than those in the bottom angles. It is important to note the amount of camber on the drawing even if special rivet spacing is provided, for it is as easy to nullify the effects of such spacing by careless fitting up or riveting as it is difficult to avoid a curve in a girder which is intended to be straight.

4. Bridge girders are often made with **curved ends** for the sake of appearance, as illustrated in Fig. 101. It is not feasible to bend both ends of long flange angles, so short angles are used at each end. These angles extend far enough horizontally so that they may be spliced to the top flange angles satisfactorily. They serve as end stiffeners, and they may be crimped over the bottom flange angles or arranged as shown.

5. **Typical drawings** of plate girders are illustrated as follows: — Figs. 98 and 99, a stringer and a floor beam for the same railroad girder bridge; Fig. 100, a floor beam for a truss bridge, showing the method of cutting the end to clear the pin; Figs. 101 and 102, railroad bridge girders; Fig. 103, a crane girder with holes in the top flange for rail-clamp bolts and for lattice bars which connect to the stiffening girder of Fig. 110; Fig. 104, a floor girder with beam connections; and Fig. 105, a box girder with two webs.

CHAPTER XIX

LATTICED GIRDERS

SYNOPSIS: Latticed girders are light trusses with parallel chords, but a different system of working lines is used from that of the next two chapters.

1. **Definition.** — The terms "latticed girder" and "latticed truss" are not distinctive because they are used interchangeably by some companies or individuals whereas they have different meanings when used by others. A girder becomes a latticed girder when the solid web is replaced by separate web members, but it also becomes a truss from the definition (page 17). Formerly, a latticed truss was of a form similar to that shown in Fig. 120 (*h*) which had from two to four independent web systems, the stresses of which were statically indeterminate. Since this type of truss is becoming obsolete except for very light work (Fig. 111), the name "latticed truss" is often used to apply to almost any form of riveted truss with parallel chords to distinguish it from a pin-connected truss. In this book the term "**latticed girder**" will be confined to comparatively light trusses with parallel chords, all members of which are composed of one or two angles. Heavier forms will be called trusses, and to avoid ambiguity the term "latticed truss" will not be used; it will be replaced by the more specific terms "Pratt truss," "Howe truss," etc., as illustrated in Chapter XXI, page 120.

2. The most **common form** of latticed girder is the "Warren" or "triangular," shown in Fig. 120. Some authorities limit the term "Warren" to girders formed of equilateral triangles, but this distinction is not generally maintained. In order to increase the number of panels, or to provide connections for other members, every triangle may be subdivided as in Fig. 120 (*k*), alternate panels may be subdivided as in Fig. 120 (*m*) and (*n*), or single panels may be subdivided as in Fig. 110.

3. **End Connections.** — The greatest number of latticed girders are used in building construction where they are supported by columns or other members. Typical column connections are shown in Figs. 110 and 111, the former to the face of a column perpendicular to the plane of the girder and the latter to the face parallel to this plane. Note that in the type of connection shown in Fig. 111, one angle of each chord must be cut short to avoid interference with the column. Small foot bridges are often supported by latticed girders which rest upon the abutments, with shoe plates similar to those used on plate girders.

4. **Proportions.** — The depth of a latticed girder is determined in the designing department, and it is usually expressed in even half feet. In building work the depth is often dependent upon other framing because the position of both the top and the bottom may be fixed by other members. The panel lengths are made equal in the drafting room, but seldom are the resulting triangles equilateral (see above).

5. In general the different members of a truss are referred to a system of **working lines.** Theoretically these working lines should pass through the centers of gravity of the different members, but when a member is composed of one or two angles it is customary to use the rivet lines as working lines. These working lines should intersect in a common point at each apex as in roof trusses (Chapter XX, page 113) and bridge trusses (Chapter XXI, page 120); the stresses are determined upon this assumption. In the case of light latticed girders, however, the stresses are usually so small that a slight deviation makes practically no difference in the efficiency of the girders, and better connections are thus

obtained. The rivet lines of the diagonals are not extended to the rivet lines of the chords but they intersect parallel auxiliary working lines; the end rivets of the diagonals are placed at these intersections, as shown in Fig. 110. The position of these auxiliary working lines should be such that ample clearance is left between the angles of the diagonals and the chord angles, as explained in detail on page 77 : 1.

1. By means of the dimensions determined in step IV (page 77 : 1) may be found the **panel depth** and the **panel length,** each measured from center to center of the end rivets of a single diagonal. The panel length is found by deducting from the extreme length of the girder the sum of the distances from the ends to the first working points and the distances between the working points in each intermediate plate, and then dividing the remainder by the number of diagonals. This resulting panel length is expressed to the nearest sixteenth inch, or preferably the nearest eighth or quarter, the amount of clearance at two or more points being changed slightly if necessary to make the proper adjustment. Care should be taken, however, to keep all panel lengths equal, and all plates alike, as far as possible, in order to minimize the number of different templets. For illustration, let us determine the panel dimensions for the girder shown in Fig. 110. The diagonal distance from the end rivet to the farther corner of any diagonal is found by means of the diagram on page 313 to be $2\frac{5}{16}''$ for a $1\frac{3}{4}''$ gage and $1\frac{1}{2}''$ edge distance. The distance from the end rivet to the rivet line of the upper chord angles is $4\frac{1}{16}'' = 2\frac{5}{16} + \frac{1}{4} + 3\frac{1}{2} - 2$, allowing $\frac{1}{4}''$ clearance (page 72 : 2); to eliminate sixteenths 4″ is used. The corresponding distance at the bottom is made the same in order to keep the intermediate plates alike. The panel depth from center to center of working lines is $7'\text{-}6'' = 8'\text{-}6'' - 2\,(4 + 2)$. The distance in the intermediate plates between the end rivets of adjacent diagonals is $4\frac{7}{8}'' = 2\frac{5}{16} + \frac{1}{4} + 2\frac{5}{16}$. Unless it is desired to make provision for assembling the diagonals with the outstanding legs on *either* the upper or lower edges, advantage may be taken of the fact that there is no plate similar to the end plate or the center plate; at each of these points the diagonals are so arranged that space need be provided only for the corners nearer the end rivets. The diagonal distance from the end rivet to the *nearer* corner is found from the diagram to be $1\frac{15}{16}''$ for $1\frac{3}{4}$ (leg minus gage) and $1\frac{1}{2}$. The distance from the end of the girder to the first working point is $5\frac{11}{16}'' = 1\frac{15}{16} + \frac{1}{4} + 3\frac{1}{2}$. From the center line to the nearest working point is $5\frac{7}{8}'' = 1\frac{15}{16} + \frac{1}{4} + 3\frac{1}{2} + \frac{3}{16}$. Considering one-half of the symmetrical girder, the amount to be divided into four equal panels is $17'\text{-}9\frac{3}{16}'' = \frac{1}{2}\,(39'\text{-}10\frac{3}{4}'') - 5\frac{11}{16} - 3 \times 4\frac{7}{8} - 5\frac{7}{8}$. This is not divisible by four within the usual working limits, but to the nearest $\frac{1}{4}''$ we can use 4 panels of $4'\text{-}5\frac{1}{4}'' = 17'\text{-}9''$; this leaves $\frac{3}{16}''$ to be distributed among the other dimensions in order to make the total length check with the overall dimension. Each of the three $4\frac{7}{8}''$ dimensions could be increased by $\frac{1}{16}''$; but it would be preferable to avoid sixteenths, and consequently the $5\frac{11}{16}''$ is increased to $5\frac{3}{4}''$ and the $5\frac{7}{8}''$ to 6″, as shown.

2. **The dimensions** should be recorded on the drawing as soon as determined. When the panel depth and length are found they may be laid out and dimensioned, and the angles may be drawn and billed; the gages should be dimensioned on each angle though the standard gage is used.

3. The sizes of the connection **plates** may be determined either graphically or arithmetically as explained on page 77 : 1, V. The experienced draftsman usually prefers the second method, particularly for rectangular plates. The size of the plate will generally be determined by the rivets in the diagonals, and afterwards the remaining rivets may be spaced. To illustrate, let us find the size of the plate *pa*, Fig. 110. On the main drawing along the working line of one of the diagonals already plotted to scale, a distance is laid off equal to the sum of the rivet spaces, 6″ = 3 + 3, from the end rivet to the last rivet in the group. A scale of 3″ = 1′ should be used, or else the lines should be prolonged so that a full size scale may be used. The corresponding vertical and horizontal components may be scaled without drawing any additional lines. In the case at hand, the vertical component is $5\frac{1}{8}''$ and the horizontal component is $3\frac{1}{16}''$. By combining these values with the edge distances and the proper distances from the second paragraph preceding, the dimensions of the plate are found. Thus, the width of the plate with $1\frac{1}{2}''$ edge distances would be $12\frac{1}{8}'' = 1\frac{1}{2} + 4 + 5\frac{1}{8} + 1\frac{1}{2}$; by reducing each edge distance to $1\frac{7}{16}$, the commercial width of 12″ may be used (see page 43 : 3). The length of the plate is $1'\text{-}2'' = 1\frac{1}{2} + 3\frac{1}{16} + 4\frac{7}{8} + 3\frac{1}{16} + 1\frac{1}{2}$. The remaining rivets in the plate are

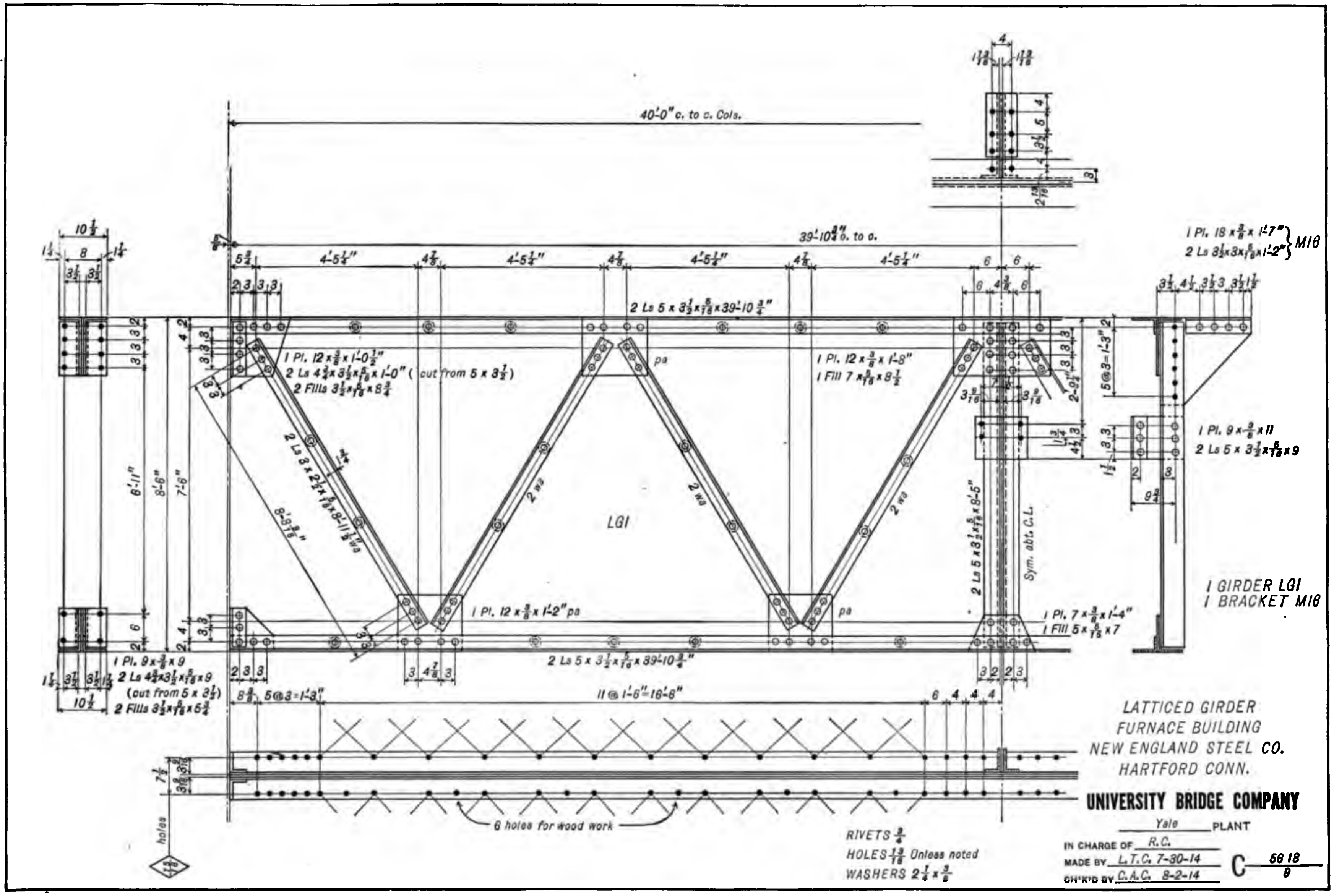


Fig. 110. Typical Latticed Girder.

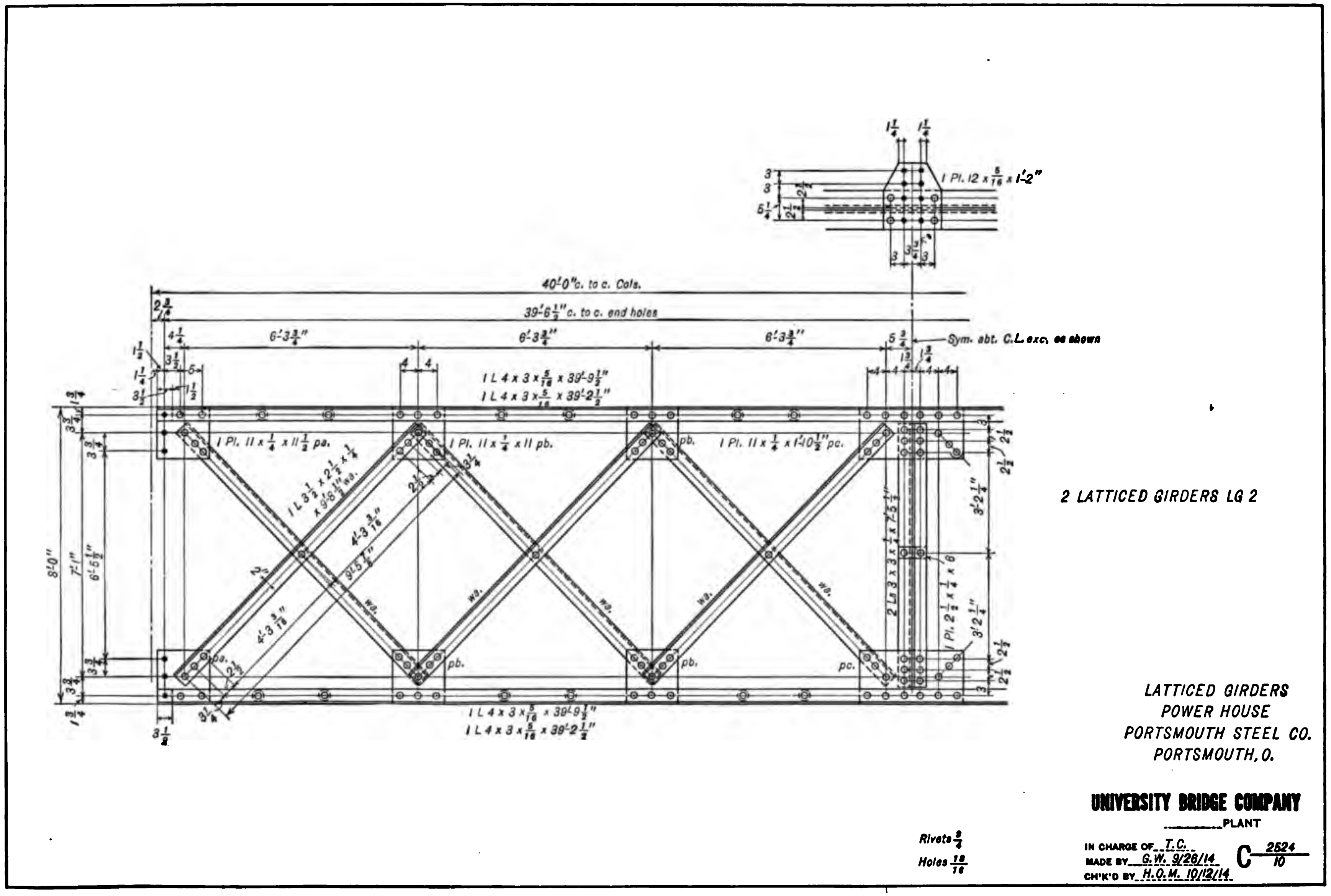


Fig. 111. Double Latticed Girder.

located with reference to the working points of the diagonals with due consideration to the edge distances and the maximum spaces allowed (page 69 : 1). For the sake of appearance the plate should be cut so that it is no shorter along the chord angles than elsewhere. In the plate referred to it is convenient to place two rivets directly below the end rivets of the diagonals in order to save additional dimensions. The remaining spaces are expressed to the nearest $\frac{1}{4}''$ so that the edge distances will be approximately $1\frac{1}{2}$ (in this case $1\frac{9}{16}$).

1. When adjacent diagonals have **different numbers of rivets,** the connecting plate may be cut diagonally, or else the smaller number of rivets may be spread so that the resulting edge distances will not be excessive. Diagonal cuts should preferably extend the full width of the plate in order to save waste; in this case the width of the plate should be the perpendicular distance between the parallel sides even though this is the longer dimension. For other suggestions regarding the shape of plates see page 76 : 1, IX.

2. The size of the **end connection plates** may be governed either by the rivets in the diagonal, or by the number of field rivets required, whether connection angles are used or not. Compare Figs. 110 and 111. In general, the rivets along the chords and along the supporting members should be so spaced that the full width and length of the plate are used; that is, the plate should not be cut so that these edges are shorter than the opposite edges.

3. When a double system of diagonals is used, as shown in Fig. 111, the two single-angle members on opposite sides of any plate may be made to overlap in order to take one rivet in common. The adjacent rivets must be spaced far enough from the edges of the angles to allow sufficient driving clearance (page 73 : 5). The angles of such **double latticed girders** should be riveted at their intersections, with or without washers.

4. All members which are composed of two angles should be fastened together by means of **stitch rivets,** spaced as explained on page 69 : 4. The spacing in a tension web member may be made the same as that in a similar compression member if by so doing the members are made alike.

5. **Typical connections** for roof trusses are shown at the centers of both *LG* 1 and *LG* 2, Figs. 110 and 111. A girt connection is also shown at the center of *LG* 1.

6. A latticed girder is often used as a "**stiffening girder.**" It is placed alongside a long-span crane girder with either the top or the bottom chord at the proper elevation so that it can be connected to the top chord of the crane girder by means of tie plates and lattice bars; in this way the crane girder is stayed against buckling under the effects of transverse thrusts of the crane due to swinging loads, etc. Thus *LG* 1 is provided with holes in the bottom chord to correspond to those in the top flange of *G* 2, Fig. 103. In this case the stiffening girder is to be placed between two crane girders, and hence has holes in both sides.

CHAPTER XX

ROOF TRUSSES

Synopsis: Directions are given for making drawings of typical trusses which support ordinary pitched roofs. "Flat roofs" are usually supported either by beams, or by trusses similar to latticed girders.

1. **Steel roof trusses** are used in mill building construction, or wherever a comparatively large area is to be covered without the use of intermediate supports. The comparatively flat roofs in office-building construction are usually supported by beams (page 90 : 1). "Flat roofs" which are pitched just enough to provide proper drainage are carried over long spans by "flat" trusses (Fig. 113 (*h*)), which resemble latticed girders except that the top chords are inclined slightly.

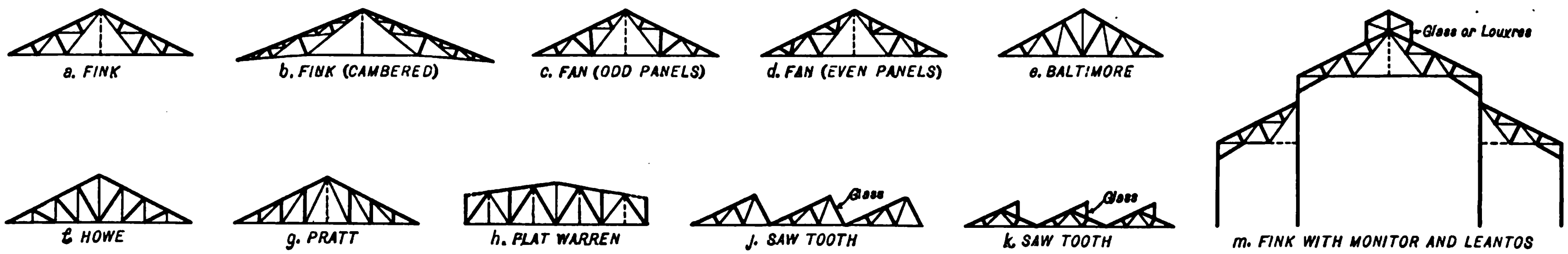


Fig. 113. Types of Roof Trusses.

2. The more **common types** of roof trusses are shown in Fig. 113. Compression members are indicated by heavy lines, and tension members by fine lines; dotted lines represent members which receive no stress from the usual loads but which are generally added to support intermediate loads, to prevent a member from sagging, or to give greater rigidity to a structure. The "Fink" truss, or a modification of it called the "Fan" truss, is well adapted to most requirements. In its simplest form it has only a single strut at the center of each top chord, but for longer spans additional panels are added, as shown at *a* and at *b*. In the Fink truss the top chord is divided into an *even* number of equal panels, and the struts are at right angles to the top chord; the number of panels may be doubled by the insertion of another strut in the center of each panel. If it is desired to increase the number of panels without doubling the number, one or more of the struts may be replaced by two members not at right angles to the top chord, and the truss becomes a "Fan" truss as shown at *c* and at *d*. The "Baltimore" truss, *e*, and the "Howe" truss, *f*, are used more for wood construction. The normal struts in the Baltimore truss divide the top chord into equal panels; since two of these struts meet at the center, the pitch of the truss cannot be assumed but it must be calculated to correspond to the length of the span and to the number of panels. Vertical members divide the "Howe"

truss, *f*, and the "Pratt" truss, *g*, into equal panels. The diagonals of a Howe truss are in compression, while those of a Pratt truss are in tension as in bridge trusses, (page 120 : 2); note that in either case the diagonals in the roof trusses and in the corresponding parallel chord trusses slope in opposite directions. Trusses may be cambered as shown at *b*, to increase the under clearance in the center of the building without a corresponding increase in the height of the walls; the horizontal chord is used much more commonly. "Saw-tooth" roofs are used to provide a more satisfactory lighting system within the buildings; the vertical, or nearly vertical portions of the roof are so placed that the northern light (in the northern hemisphere) can pass through the glazed surfaces, the direct sunlight being excluded. Ventilation or light is often obtained by raising the central portion of the roof by means of a "monitor" or "clearstory" as shown at *m*.

1. **The "pitch"** of a symmetrical roof truss is the ratio of the rise to the span. The slope of the roof or tangent of the angle between the top and the bottom chords is not equal to the pitch, but to *twice* the pitch. Thus the slope of a ¼ pitch roof is 6 in 12. ¼ pitch trusses are used most extensively, other common pitches being ⅙, ⅓, and 30°.

2. **Purlin Spacing.** — The form of the truss and the approximate spacing of the purlins are generally determined in the designing department so that the purlins and the trusses can be properly designed. The purlins are the longitudinal members which support the roof. If possible, the purlins should be placed near the panel points of the trusses to avoid flexure in the top chords; the designer must anticipate the spacing with sufficient accuracy so that he can determine the proper number of panels and ascertain whether the top chord must be designed for combined compression and bending. The exact spacing of the purlins is left for the draftsman; it necessarily depends upon the style of the roofing. If tiles rest directly upon the purlins the spacing must conform to the lengths of the tiles. For any form of roofing which is supported by wooden sheathing, the purlins should be so placed that commercial lengths of lumber may be used without excessive waste. Commercial lengths of lumber are usually multiples of 2 feet. The roofing which is most used for mill buildings is corrugated steel, and when this rests directly upon the purlins the latter must be placed at such intervals that sheets of standard lengths may be used. Corrugated steel sheets vary in length by multiples of 6 inches from 48 to 126 inches. As far as possible the sheets extend over two purlin spaces.*

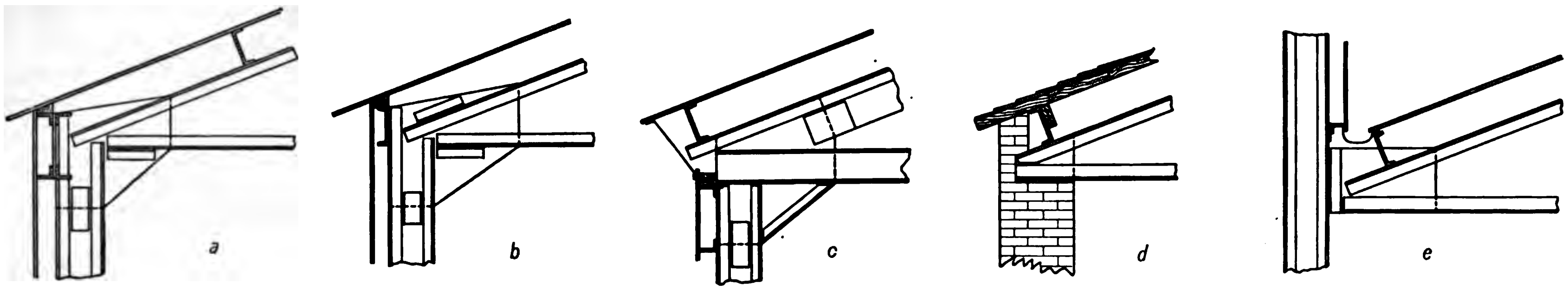


Fig. 114. Types of Heel Connections.

3. Roof trusses may rest directly upon masonry walls or they may be supported by girders or trusses, but more frequently they are riveted to columns. The type of heel connection depends not only upon the form of support but also upon the detail of the cornice at the eaves; this in turn is dependent upon the style of the roof covering and of the

* For a fuller treatment of different roofing materials and for more detailed information regarding corrugated steel see Ketchum's "Steel Mill Buildings" or "Structural Engineers' Handbook," or Tyrrell's "Mill Buildings," McGraw-Hill Book Co., Inc., New York.

side walls. Some of the more usual forms of heel plates are illustrated in Fig. 114; the different features shown often occur in other combinations.

1. **Arrangement on Sheet.** — The members of a roof truss are drawn in the same relative position which they occupy in the completed structure. This is usually done even though the truss is shipped "knocked down," i.e., each chord and web member shipped separately. If a truss is symmetrical about the center, or nearly so, only one-half need be shown on the drawing. A typical drawing for a roof truss is shown in Fig. 116.

2. **A system of working lines** is first laid down upon the drawing, and all dimensions are referred to these lines. These lines represent approximately the lines of stress of the different members, and at each apex the lines should meet in a common point (compare page 108 : 5). The rivet lines of angles are almost always used as working lines instead of lines through the centers of gravity; when two rivet lines are used in one leg, the one nearer the back of the angle is chosen. The working lines may be laid down to scale as soon as the effective length and the rise (or the pitch) are known. The panels may be made equal by means of any convenient scale, and all working lines may be drawn even if the lengths are unknown. The effective length of span is the distance between the centers of the columns or the centers of the bearings. A scale of $\frac{3}{4}'' = 1'$ (or $1'' = 1'$), is desirable for the details, but if a truss is too large to permit the use of the same scale for the working lines, they may be plotted to a smaller scale (a decimal scale if desired). As soon as the working lines are plotted the corresponding dimension lines should be drawn; these lines should be so placed that they will not interfere with each other or with the details which are to be added later. Usually an experienced draftsman can anticipate the final position of these lines; the beginner should use other drawings as a guide, and arrange the lines to the best of his ability even though some of them may have to be changed later. It is important to have dimension lines upon which to record the computed dimensions between the working points; these dimensions should then be determined and recorded upon the lines. The computation may be simplified by the comparison of similar triangles; thus for example, if the half-span and the rise of the Fink truss shown in Fig. 115 are known, all but two lengths may be found by proportion.

In this truss the total slope distance or hypotenuse C should be found from A and B by means of a table of squares (or from A and the pitch by logarithms). This distance should be divided into four equal parts; if not equally divisible, some of the panels may be made $\frac{1}{16}''$ longer than the others in order to avoid thirty-seconds and yet maintain the proper total. The strut of length D bisects the top chord and forms two equal right triangles, one vertex being at the heel and the other at the peak. Each of these triangles is similar to the original half-truss for the angles are equal; therefore the length D bears the same relation to its base $\frac{C}{2}$ that the rise B bears to the half-span A. From D and $\frac{C}{2}$ the hypotenuse E may be obtained by squares or by logarithms. The remaining distances may be found as indicated on the figure.

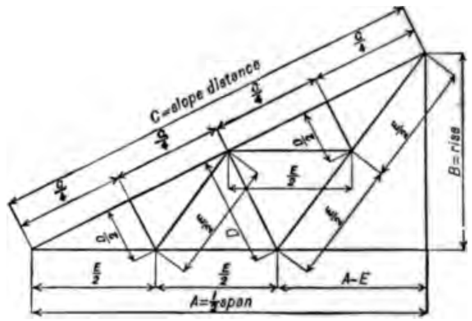


Fig. 115.

It is better to compute the sides of the larger triangles and to get the sides of the other similar triangles by *division*, rather than to compute the shorter lengths and obtain the others by multiplication. In this way the computed lengths may be taken to the nearest $\frac{1}{16}''$ and the corresponding shorter lengths will necessarily result within $\frac{1}{16}''$ of the true values.

3. **Form of Members.** — The top chord of a roof truss is usually composed of two angles with the outstanding legs along the upper edge. In order to stiffen the truss laterally the longer legs are often outstanding, the shorter legs being vertical. When the purlins cannot be placed at panel points of the truss, the top chord must be designed to resist bending as well as direct compression; in this case either the longer legs are placed vertically or else a web plate is inserted to form a T-shaped member, as shown in Fig. 114 (*c*). The bottom chord is usually composed of two angles with the longer legs vertical, the outstanding legs being along the lower edge. Two channels are sometimes used if the bottom chord is designed to support a small hoist or other direct load

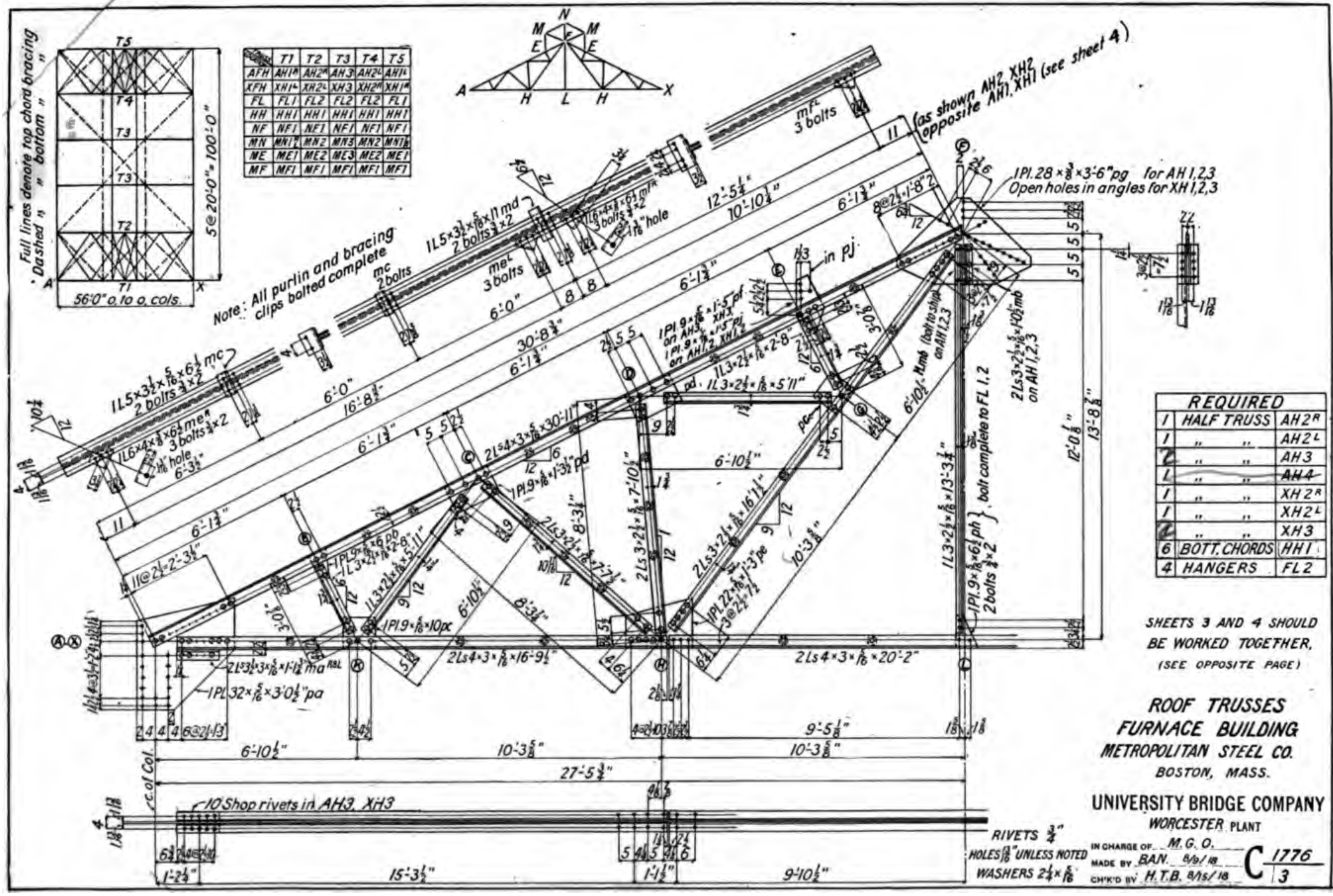


Fig. 116. Typical Drawing for a Roof Truss.

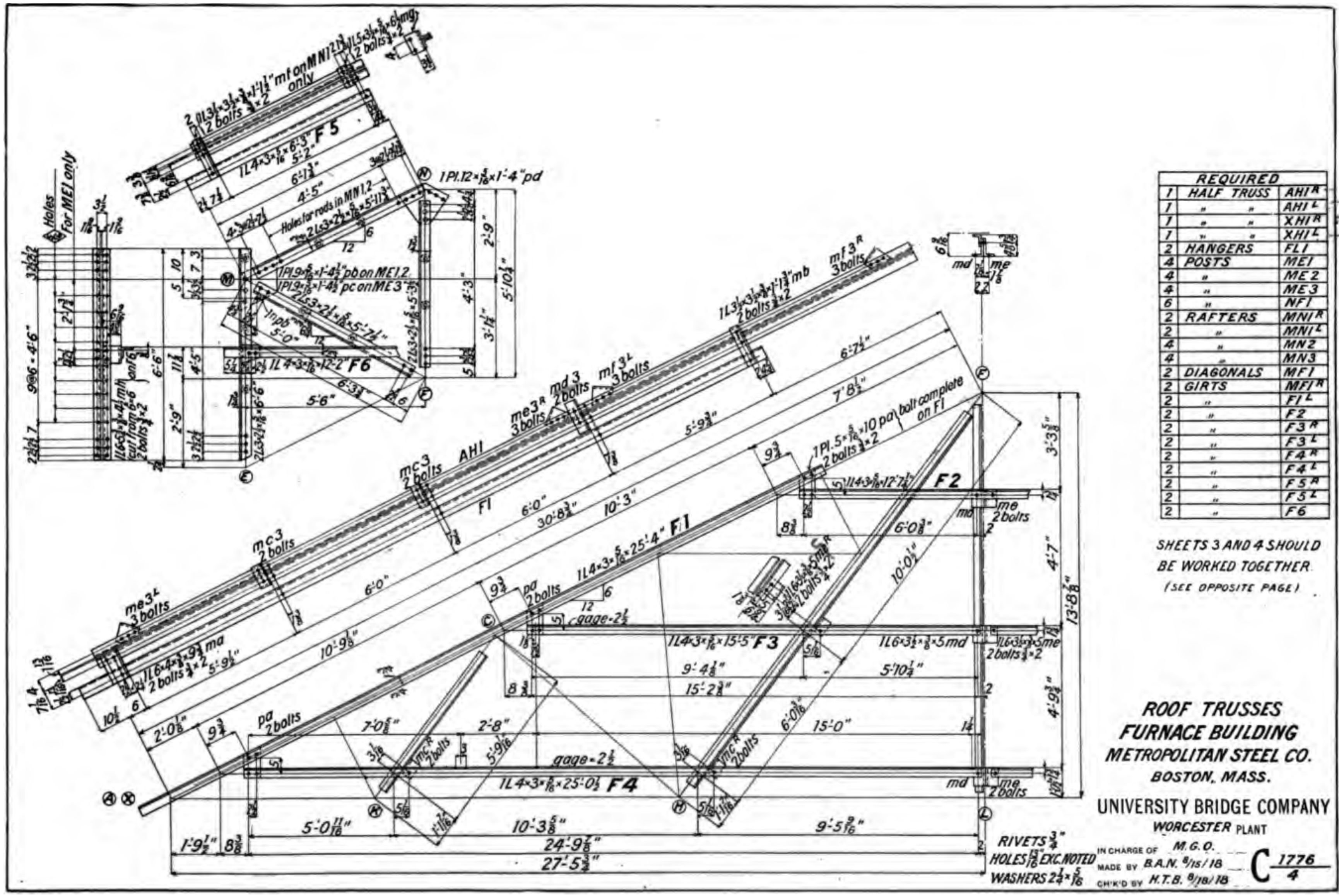


Fig. 117. Gable Girts and Monitor Framing.

The web members are composed of one or two angles with the longer legs vertical. The outstanding legs may be along either edge, but for the sake of appearance some systematic arrangement should be adopted. More frequently the backs of the angle are downward because the members appear to be stronger when viewed from below; as a matter of fact the compression members are not so strong, although the difference is negligible. It is sometimes desirable to turn one member with the back on the opposite edge if by so doing two members or two connection plates may be made alike.

1. The different members of a truss are connected by means of **gusset plates.** The number of rivets required in each member is determined from the stress in the member, as explained on page 233 : 2. The shape and the dimensions of the gusset plates are determined graphically by means of layouts, as explained on page 76 : 1. When a continuous web plate is used between the angles of the top chord a web member may be connected directly to this plate without the use of gusset plates, provided there is room for the proper number of rivets; otherwise, part of the web plate must be replaced by a gusset plate, suitable splices being used to connect them. The size of a gusset plate may sometimes be reduced if the outstanding leg of an angle is connected to the plate by means of a "lug" or connection angle as shown in Fig. 118 (*a*). One leg only is connected as a rule unless the number of rivets exceeds 7 or 8.

2. Each truss member which is composed of two angles should have the angles fastened together by **stitch rivets,** as explained on page 69 : 4. Members composed of two channels are fastened similarly by pairs of rivets with a $2\frac{1}{2}''$ or $3''$ bar between, instead of a washer, as shown in Fig. 118 (*a*). A careless mistake quite common among draftsmen and tracers is to show stitch rivets in members composed of single angles. This is not a serious blunder, but unless it is detected in the templet shop it may cause the punching of extra holes.

3. **Center Hanger.** — A light auxiliary member or "hanger" is often used as an intermediate support for the longer bottom-chord members even though no corresponding stress results from the usual loads. Such members are shown by dotted lines in many of the trusses shown in Fig. 113. The purpose of these members is twofold. During erection a truss is assembled on the ground and then raised into its final position as a whole, without falsework, by a locomotive crane or a gin pole; during this stage the bottom chord is in compression and it may buckle unless the long center chord member is stayed. After the building is completed workmen are likely to attach block and tackle to the bottom chord for the purpose of lifting heavy loads, even though the truss is not designed for this purpose; this is less serious if the long center panel is subdivided. A single light angle is used as a center hanger, with the rivet line at the center of the truss unless it is offset to provide a connection for a ridge strut (see below). In either case the connection at the bottom should be made so that the bottom chord member is symmetrical about the center line to simplify erection.

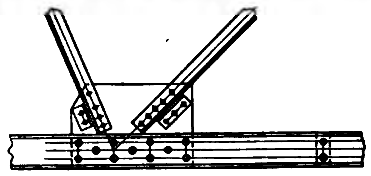


Fig. 118 (*a*).

4. **Shipment.** — A roof truss is usually riveted in the shop completely, or else in as large sections as can be shipped; for export each member is usually shipped separately. The maximum height which can be shipped by rail is about 10′-6″. If the center height exceeds this amount the truss is shipped in sections, as shown in the diagram in Fig. 116. Each half-truss is shipped on the top chord as a base; if the maximum normal distance exceeds 10′-6″, smaller sections must be made. For the method of marking, see page 82 : 2.

5. **A ridge strut** must be provided as the compression member of the top-chord bracing system, unless purlins at the peak serve the same purpose.

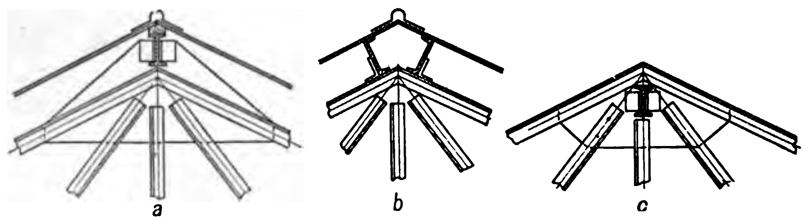


Fig. 118 (*b*). Details at the Peak.

In *a* and *b*, Fig. 118 (*b*), are shown two forms of purlins which also act as struts. When the central portion of the roof is raised to the top of a monitor (Fig. 113 (*m*)) there are no purlins at the peak of the main roof and a ridge strut must be provided. An I-beam may be used (*c*, **Fig.**

118 (*b*)), but more frequently a two-angle member similar to *S* 5 inverted (Fig. 147) is employed. A continuous line of ridge struts is used for the full length of a monitor; when two angles are used in the braced bays either one or two angles are used in the intermediate unbraced bays. The connection plate which projects from between the angles of the ridge strut is inserted between two connection angles on the peak plate of the truss. The center hanger may serve as a connection angle on one side of the truss, as shown in Fig. 116; when so used it is offset so that the strut is in the center of the truss.

1. **Purlins are connected** to the top chord of a truss by means of connection angles as shown on page 315; the larger purlins have a flange connection as well. Angles, channels, and Z-bars are stronger if the upper flanges or legs are turned up the slope. Channels are usually reversed when wooden spiking pieces are used. Purlins are usually bolted in place, and accordingly the connection angles are bolted to the truss, as noted in Fig. 116. Purlins which act as struts are connected by more bolts than the others; the strut connection is used not only in the braced bays but for the full length of the building. In Fig. 116 the purlin at panel point *D* is a strut purlin; as shown in the plan, cross bracing is used between this purlin and the ridge strut, and also between this purlin and the eave strut. For convenience, the distance between rivet lines in the top chord is made a multiple of $\frac{1}{4}''$ even though special gages are used in the angles; a single rivet line is used in each angle.

2. **Bracing rods** may be attached to the top chords by angle clips, as shown in Fig. 116; these angles should be so placed that the rods will pass through one leg at right angles in order that the nuts will bear properly. Rods of minor importance may be bent to pass through holes in the gusset plates; holes are shown in plate *pb* (Fig. 117) and in *pj* (Fig. 116) for the rods in the sides of the monitor. Similarly, extra holes are punched in the top chord angles of the monitor for rods. Straight rods may pass through slotted holes in the gusset plates provided beveled washers (shown on page 316) are used to give proper bearing for the nuts. Clevises (page 316) are not used as extensively as formerly, because other types of connection are more economical.

3. Holes must be provided for the **bottom-chord bracing,** as shown in Fig. 140. The plates are connected to the under side of the angles; when a connection angle is used at the heel, the plate is placed either between the connection angle and the bottom-chord angle or else on top of the latter. The bracing plate which connects to the truss at panel point *H* serves also as a splice plate for the bottom chord, and the number of rivets in the vertical legs may be reduced accordingly; a suitable plate must thus be provided for each truss even though there are no diagonals to be connected at this point.

4. So many buildings are extended after they have been completed, even though such extension was not anticipated at the time of construction, that it is advisable to make certain provisions for **future extension** whether specified or not. During the author's experience one building in which no provision was made for extension, was extended three times before it was completed once. If the roof truss at the end of a building is made like an intermediate truss it does not have to be moved when the building is extended. If a special end frame is made with rafters supported by columns, not only the frame but at least one panel of roof and side covering must be removed. If future extension is expected, holes should be provided to facilitate such extension. It is often as cheap to punch a few extra holes in one member in order to make it like another, as to save the extra holes by means of special notes which complicate both the drafting and the shop work.

5. If the ends of a building are to be covered with corrugated steel, girts must be placed on the **gable end** to support it. When an end truss is used, the gable columns extend only up to the bottom chord and the gable girts must be attached to the truss. These girts are shown in position, and on simple drawings they may be added to the drawing of the truss, as in the monitor, Fig. 117. In case the drawing would be too complex if this method were adopted, a second drawing may be used in conjunction with the first. Thus in Fig. 117 the working lines of Fig. 116 are reproduced and the girts are superimposed in the proper position. The truss members to which girt connections are to be added are shown in outline, and the new connections are detailed. The two drawings are worked together and a note to that effect should be added to each drawing. The required list on the second sheet should contain those members which are made wholly or in part from that drawing, while the list on the first sheet should contain the members which are made entirely from that drawing.

6. Small holes for **louvres** are provided in the sides of the monitor.

CHAPTER XXI

BRIDGE TRUSSES

SYNOPSIS: Types of trusses are shown and practical suggestions are given for making drawings of bridge trusses.

1. **When Used.** — Trusses are used in bridges for which plate girders are not well adapted either on account of the length of the span or because of economic reasons. The limiting length of plate girders is approximately 130 feet, but truss bridges are often used for spans of 100 feet or less as well as for longer spans.

2. The most common types of trusses for ordinary bridges are shown in Fig. 120. Cantilever, suspension, swing, and lift bridges are outside the scope of this book. The compression members are represented in the figures by heavy lines and the tension members by fine lines. Dotted lines represent members in which no direct stresses result from the loads indicated: thus for example, the "counters" in (*a*) to (*g*) inclusive are stressed only when a portion of the bridge is loaded, the "collision struts" in (*c*) and the posts in (*k*) are used to give intermediate support to long compression members, and other members are inserted simply to stiffen the structure or to hold other members in place. The term "Warren" is usually applied to trusses in which both the main tension and compression web members are inclined, forming isosceles triangles with the chords, as in (*i*) or (*j*). The most favorable slope of diagonal is 45°; when the panel lengths exceed 25 or 30 feet they may be subdivided, as shown in (*k*), (*m*), or (*n*). The Warren truss is used for comparatively

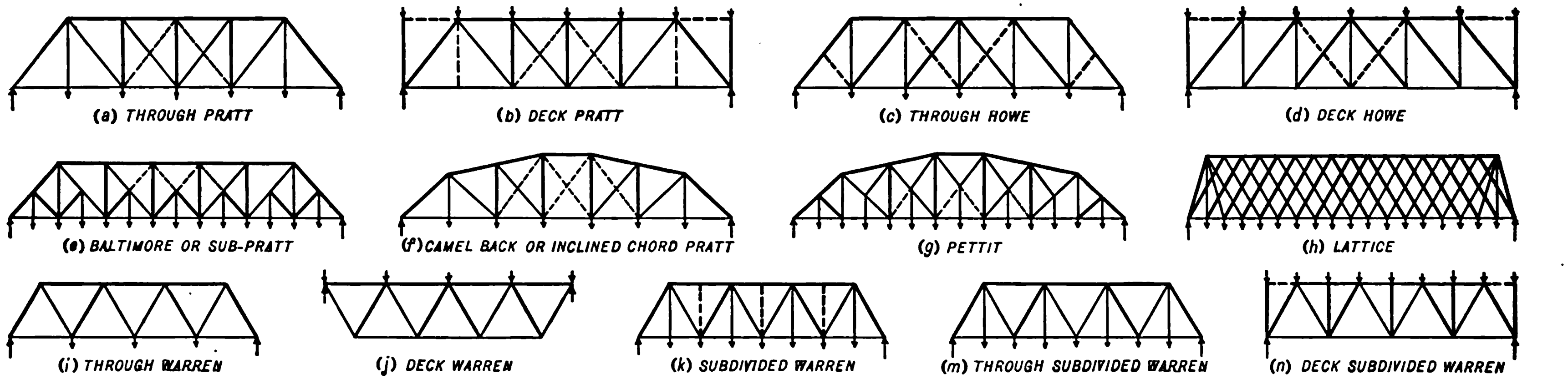


Fig. 120. Types of Bridge Trusses.

short deck bridges, and for through bridges of spans from 100 to 200 or 250 feet. A "deck" bridge is one in which the floor loads are applied at the upper chords and a "through" bridge is one in which the floor loads are applied at the lower-chord apices, and the trains pass "through" the bridge between trusses. The "Pratt" truss with parallel chords, (*a*) or (*b*), is used for spans from 100 to 175 feet, but for longer spans the "inclined chord Pratt" or "camel back" truss (*f*) is preferred in order to keep the stresses in the chords more nearly equal. For spans longer than 300 or 350 feet it is economical to subdivide the panels by verticals which extend to the middle points of the diagonals; auxiliary half diagonals are added either below the center (*e*) or above the center (*g*). If the chords are parallel, either type of "Sub-Pratt" is termed a "Baltimore" truss, but if the top chord is inclined the truss is called a "Pettit." The "Howe" truss (*c*) or (*d*) is used for wooden trusses, but it is not so well adapted to steel bridges as the Pratt truss because the compression members (the diagonals) are longer than the tension members. The "Latticed" truss (*h*) was formerly used for covered wooden bridges but it is not well adapted to steel construction; the stresses are statically indeterminate.

1. **The joints** of bridge trusses may be either riveted or pin-connected. Only one pin is used at each joint; a pin is virtually a large bolt designed as a cylindrical beam (page 278 : 2). Riveted joints are used for spans up to about 200 feet in length, particularly on railways, for the sake of economy, rigidity, and durability. Pins are used at the joints of longer spans because the secondary stresses which result from riveted joints are less easily accounted for. Often the intermediate joints of the top chord are riveted, even though pins are used in the end posts and all other members.

2. **Arrangement on Sheet.** — The smaller and lighter riveted trusses may be drawn with the members in position in order to save the duplication of details, even though the members are to be shipped separately, as illustrated in Fig. 122. More than one sheet may be required in order to show all of the necessary members; these sheets should be used to supplement each other and each should bear a note similar to that in Fig. 122. Reference points or lines may be used to indicate where the dimensions of one sheet end and those of the next sheet begin. For example, the connections at panel points $L\,2$ and $U\,2$ are fully detailed in Fig. 122; on the next sheet these panel points would be repeated and the working lines and the principal dimensions would be made to extend to these points. Enough of the gusset plates and other details should be shown in outline so that the extent of each dimension can be identified, but beyond this no attempt should be made to duplicate details which are completely shown on the first sheet. If necessary a reference line may be drawn on each sheet similar to the line X-X, Fig. 101. Each member of the larger trusses is detailed separately to avoid crowding. All vertical members are preferably drawn vertically if the size of the sheet will permit, and all horizontal members are drawn horizontally, i.e., lengthwise of the sheet; riveted diagonal members are usually drawn either vertically or horizontally in order to save space. Eye bars are drawn on small sheets or printed forms (page 174 : 2). For the sake of uniformity the members of the *left* half of the truss on the *far* side of a bridge are shown on the drawings.

3. When members are drawn in position great care must be taken to adopt the best **arrangement of views** and of dimension lines to avoid unnecessary crowding. The position of the main elevation view of each member is necessarily determined as soon as the working lines are laid down. The proper relation between views must be maintained, but views other than the front views may generally be placed so that they will not interfere with any other member; when this is not practicable a view should be so placed that only unimportant portions of a view need be omitted. For example, the side view of $L\,1$–$U\,1$ (Fig. 122) is so similar to that of $L\,2$–$U\,2$ that it is combined with it by simply adding an extra line of dimensions and the necessary notes. If a separate view was necessary it might be drawn at the right of the corresponding front view, part of the member $L\,2$–$U\,1$ being omitted where necessary. A drawing may be made clearer oftentimes if one or both ends of a view are offset from the true projection, provided that such offset is clearly indicated much as the center line through $P\,6$ (Fig. 143) is offset for $P\,7$. Space may be saved, when no ambiguity is likely to result, by combining a top view and a bottom sectional view, as illustrated in the end post and the top chord in Fig. 122; only one half of each view is shown, both views being symmetrical about the longitudinal center line.

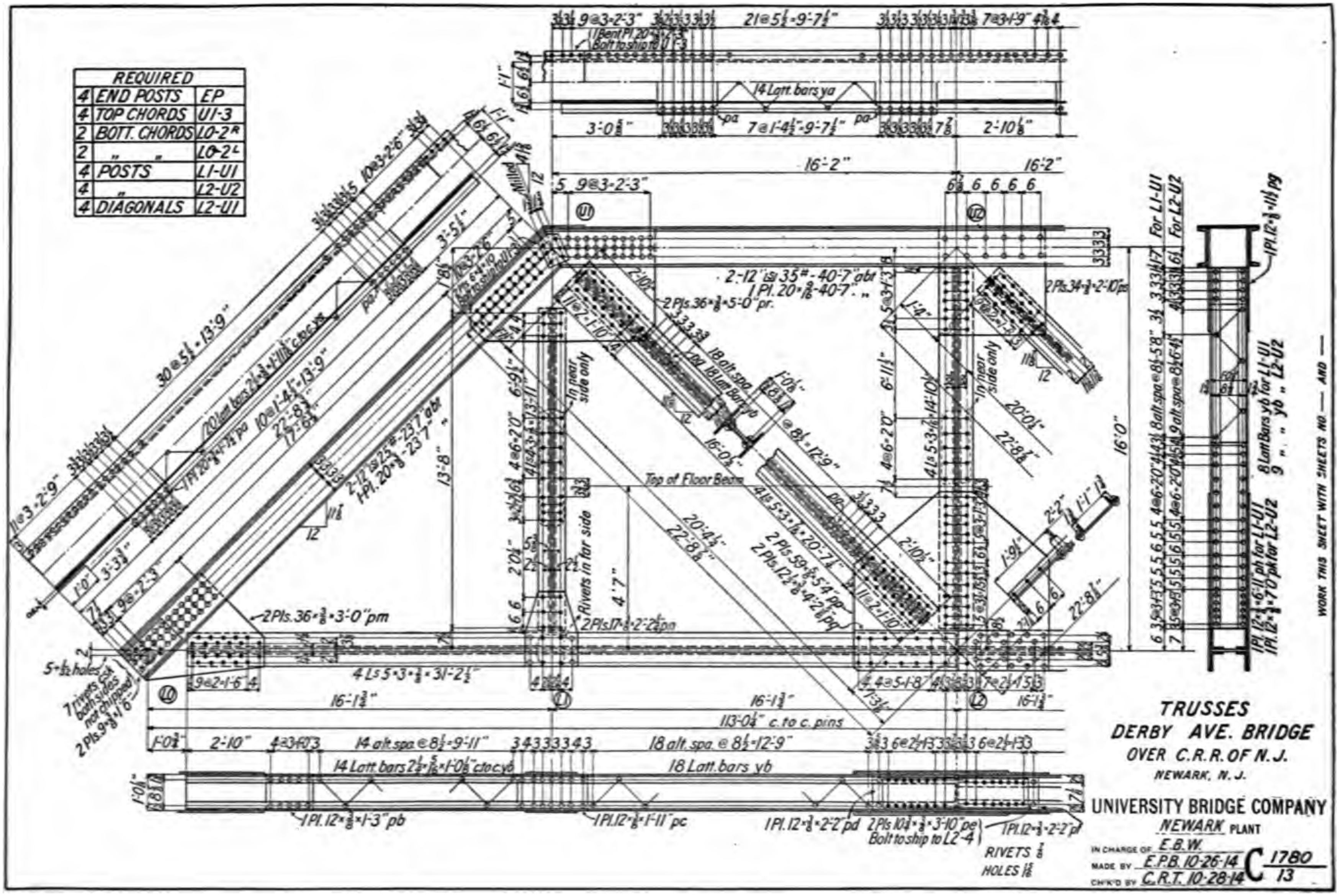


Fig. 122. Pony Truss for Highway Bridge.

1. **Shipping Marks.** — Members of bridge trusses are identified by the marks of the panel points between which they extend, as explained on page 82 : 1. It is convenient to have a small key diagram on each sheet to show the location of each member detailed on that sheet, as illustrated in Fig. 125 and 128.

2. **Camber.** — Bridge trusses are slightly arched or "cambered" so that they will assume the desired form under full load. The amount of camber should equal the maximum deflection so that the track will approach a straight line as the load is applied. The position of the panel points should be based upon the amount of deflection, and the lengths of members should be determined accordingly. For long spans the deflection should be worked out accurately,* but for spans up to about 300 feet approximately the same results may be obtained by making the lengths of the top chord members slightly greater than the lengths of the bottom chord members. Instead of both shortening the lower chord and lengthening the upper chord, it is more convenient to make the bottom panel lengths equal to the quotient found by dividing the effective span by the number of panels, and to increase the upper panel lengths enough to provide for the combined change of length in both chords. The amount of this increase for horizontal top chords or for the horizontal components of inclined top chords is $\frac{1}{16}$ inch for every 5 feet. The mean panel lengths are used in finding the lengths of the diagonal members. See next paragraph.

3. In pin-connected bridges it is well to note the **size of the pin holes,** as in Fig. 127, in order to show the size of the pins as well as the amount of clearance allowed for driving the pins. This clearance is usually $\frac{1}{32}$ (or $\frac{1}{64}$) of an inch and it should be considered in determining the lengths of the chords and diagonals. Thus if the computed length of a *compression* member falls about midway between sixteenths the next *larger* sixteenth should be chosen. The lengths of eye-bar *tension* members may be dimensioned to thirty-seconds, the full clearance of $\frac{1}{32}$ being *deducted* from the calculated length.

4. **Types of Members.** — The usual forms of top-chord or end-post members are composed either of two channels and a cover plate as in Fig. 122, or of two webs, four angles, and a cover plate as in Fig. 124. A cover plate is used on the top to furnish protection from the weather, but batten plates and lattice bars are used on the bottom to simplify the connection of the web members. A common form of bottom chord for light trusses is made of four angles latticed, as in Fig. 122; for heavier trusses four angles with side plates are used, as in Fig. 125. A solid horizontal web plate is impractical because proper drainage cannot be provided; tie plates may be used as in Fig. 125, or batten plates with lattice bars as in Fig. 122. Riveted diagonals and posts may be made of four angles, with a web or with lattice bars, as in Figs. 126 and 122. Vertical posts and hangers should have solid webs opposite the floor-beam and top-strut connections, even though lattice bars are used for the remaining portion. Posts of Pratt trusses are often made of two channels latticed. When single latticing is used on opposite faces of a member the bars should alternate, as shown in Fig. 129. Batten plates and lattice bars should be so placed that ample room is left for driving all field rivets. One or two bars may have to be shipped bolted, as in Fig. 129, so they may be removed temporarily to facilitate driving the rivets. For the design of lattice bars, see page 216 : 4; for the spacing of them, see page 70 : 1.

5. **Milling.** — The ends of chord members and end posts are milled. This is especially important in trusses with riveted joints in order that stresses can be transmitted by direct bearing. At pin-connected joints clearance of about $\frac{3}{8}$ is left between the milled surfaces to permit independent action of the members about the pins. When the members are not in the same straight line a mitered joint is used, the milled surfaces bisecting the angle between members. The slope of each mitered joint should be given to the nearest 32nd of an inch instead of the usual 16th, and also the corresponding angle should be expressed in degrees and minutes to facilitate the setting of the milling machines in the proper position.

6. **Splices.** — The riveted chords of parallel chord trusses are usually spliced independently of the gusset-plate connections of the web members, in order to avoid complications which might arise if the web members and the floor beams were connected at points where the chords change section. The splices are placed as near the gusset plates as feasible, and logically on the sides of the smaller stresses (Figs. 124 and

* See Kirkham's "Structural Engineering," McGraw-Hill Book Co. Inc., New York.

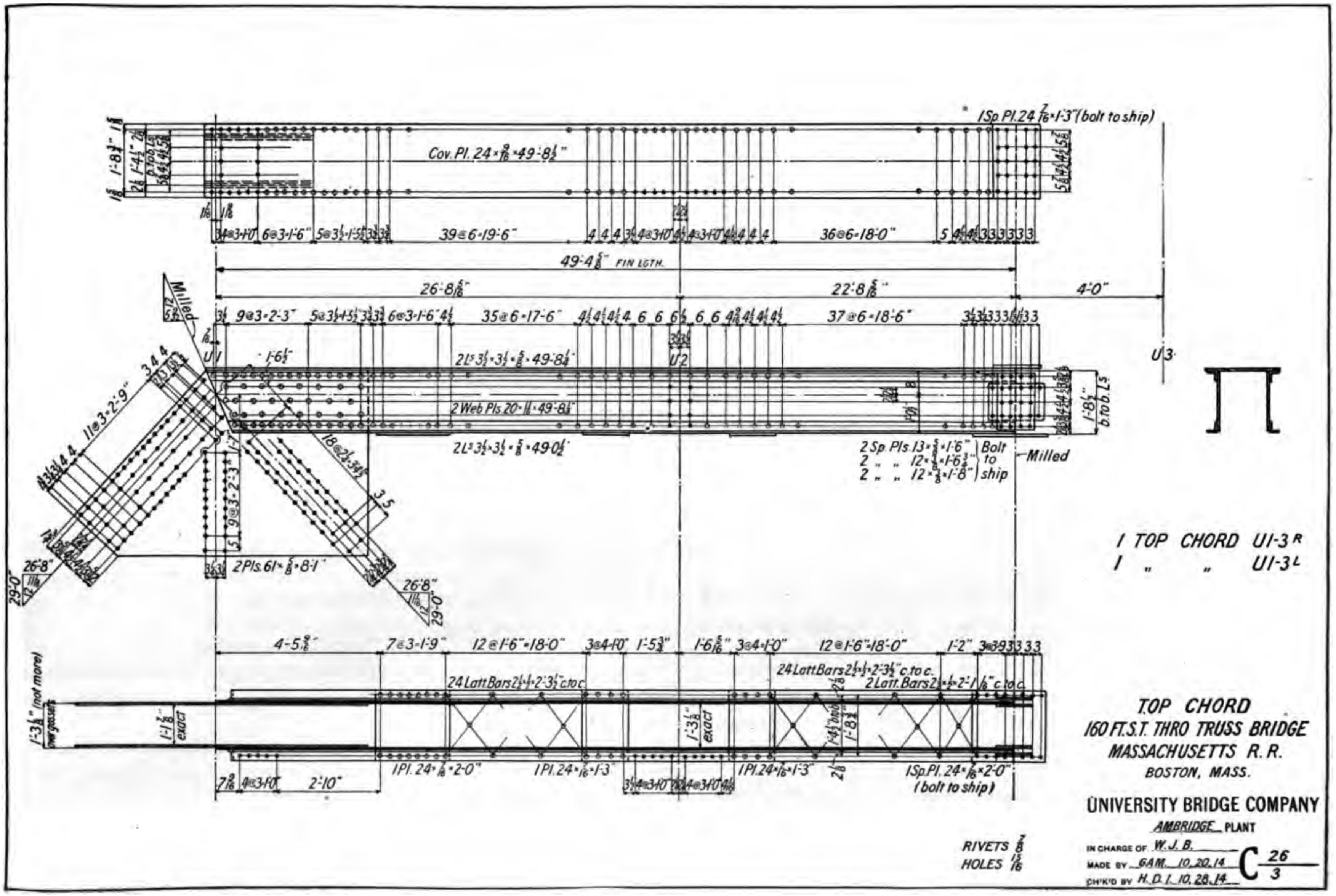


Fig. 124. Top-chord Member for Riveted Truss Shown on Next Page.

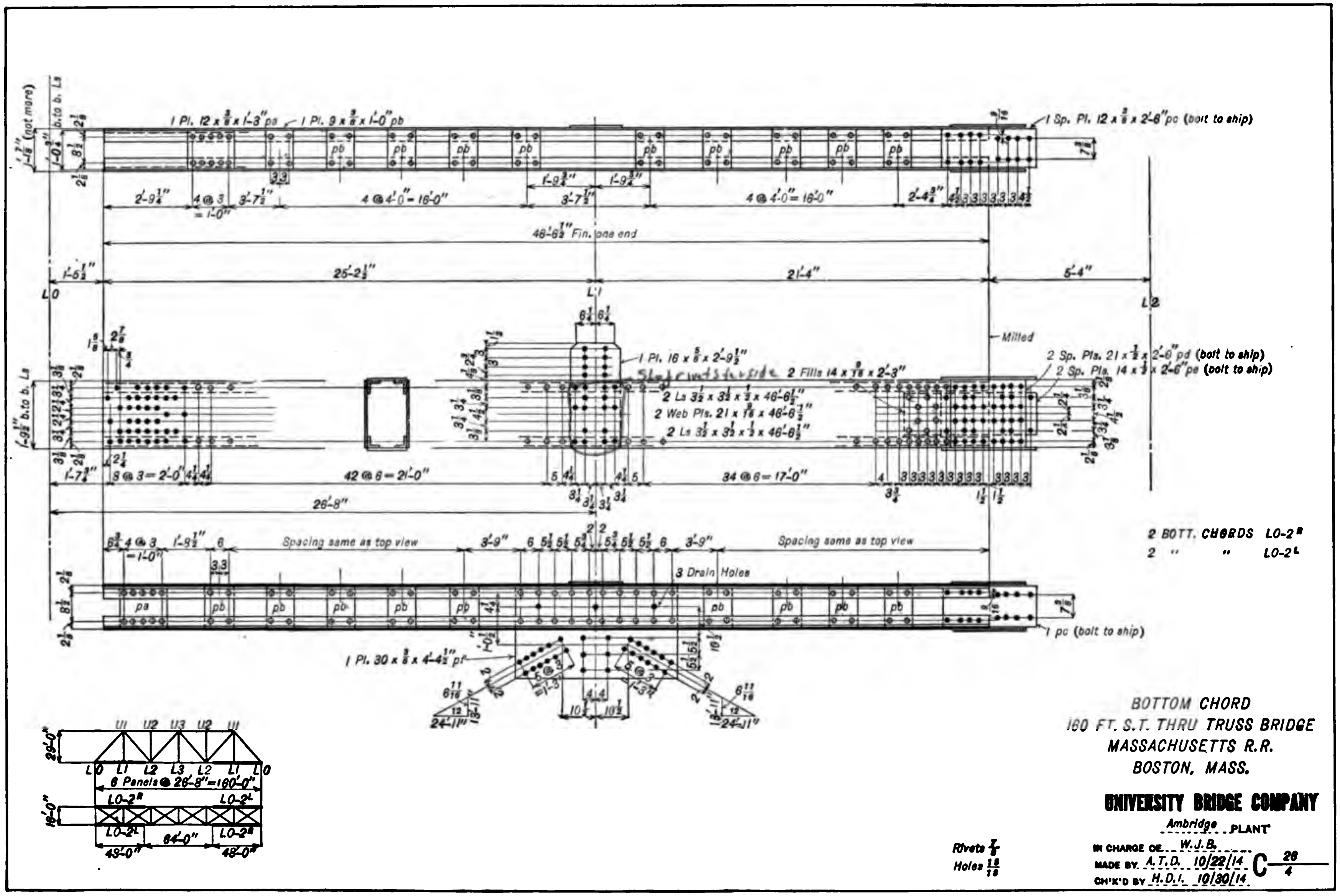


Fig. 125. Bottom-chord Member for Riveted Truss of a Railroad Bridge.

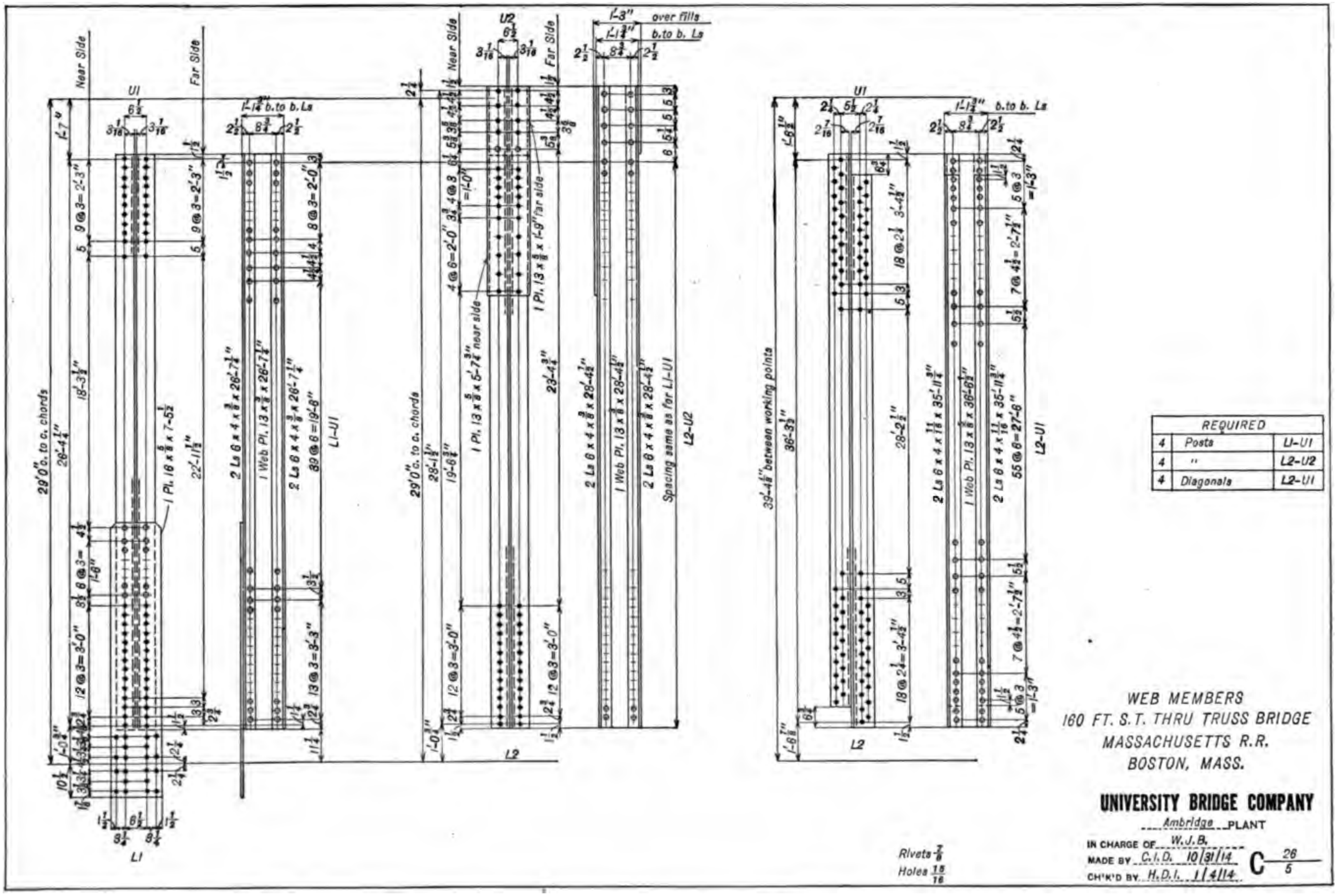


Fig. 126. Web Members for Riveted Truss Shown on Preceding Page.

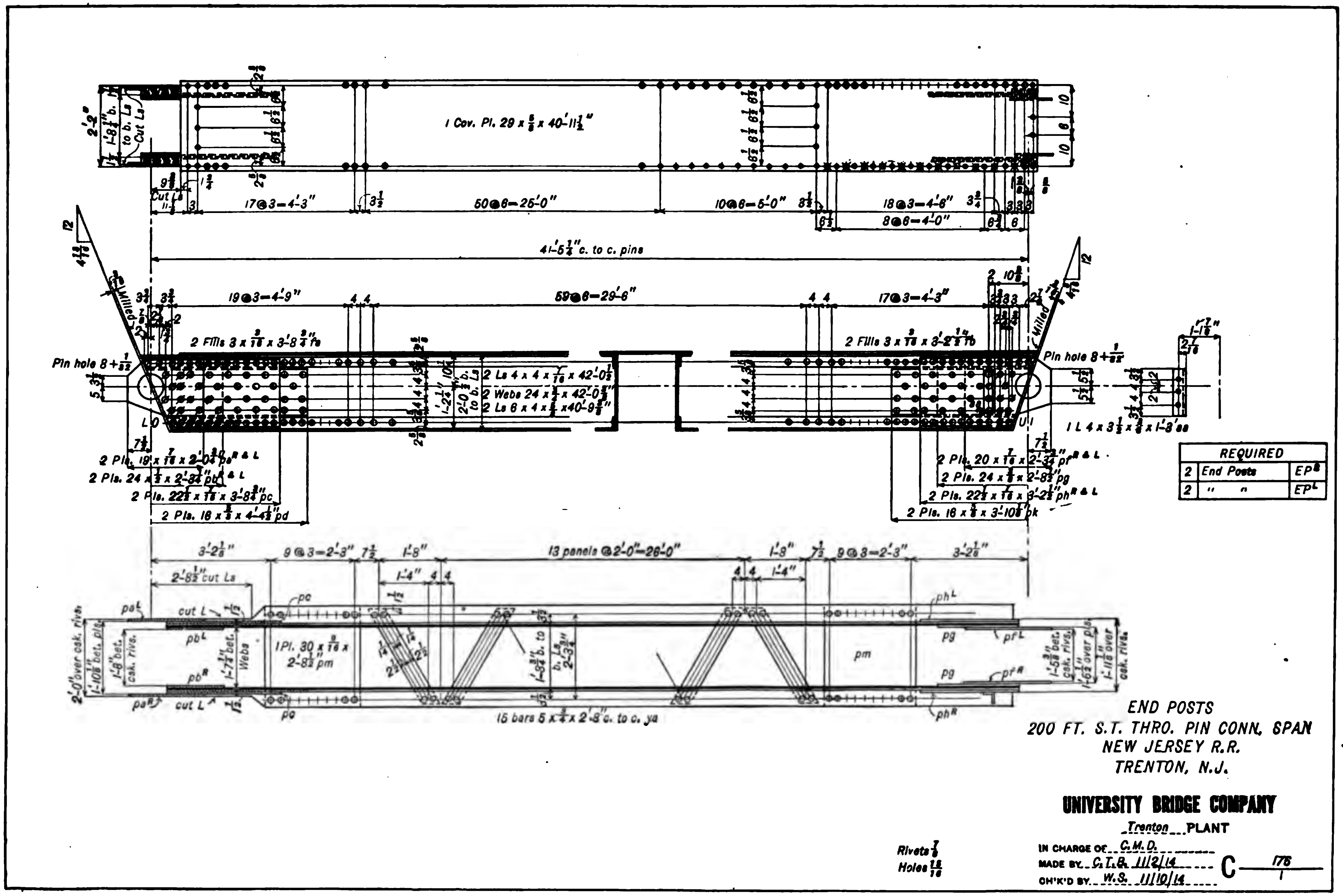


Fig. 127. End Post for Pin-connected Truss Shown on Next Page.

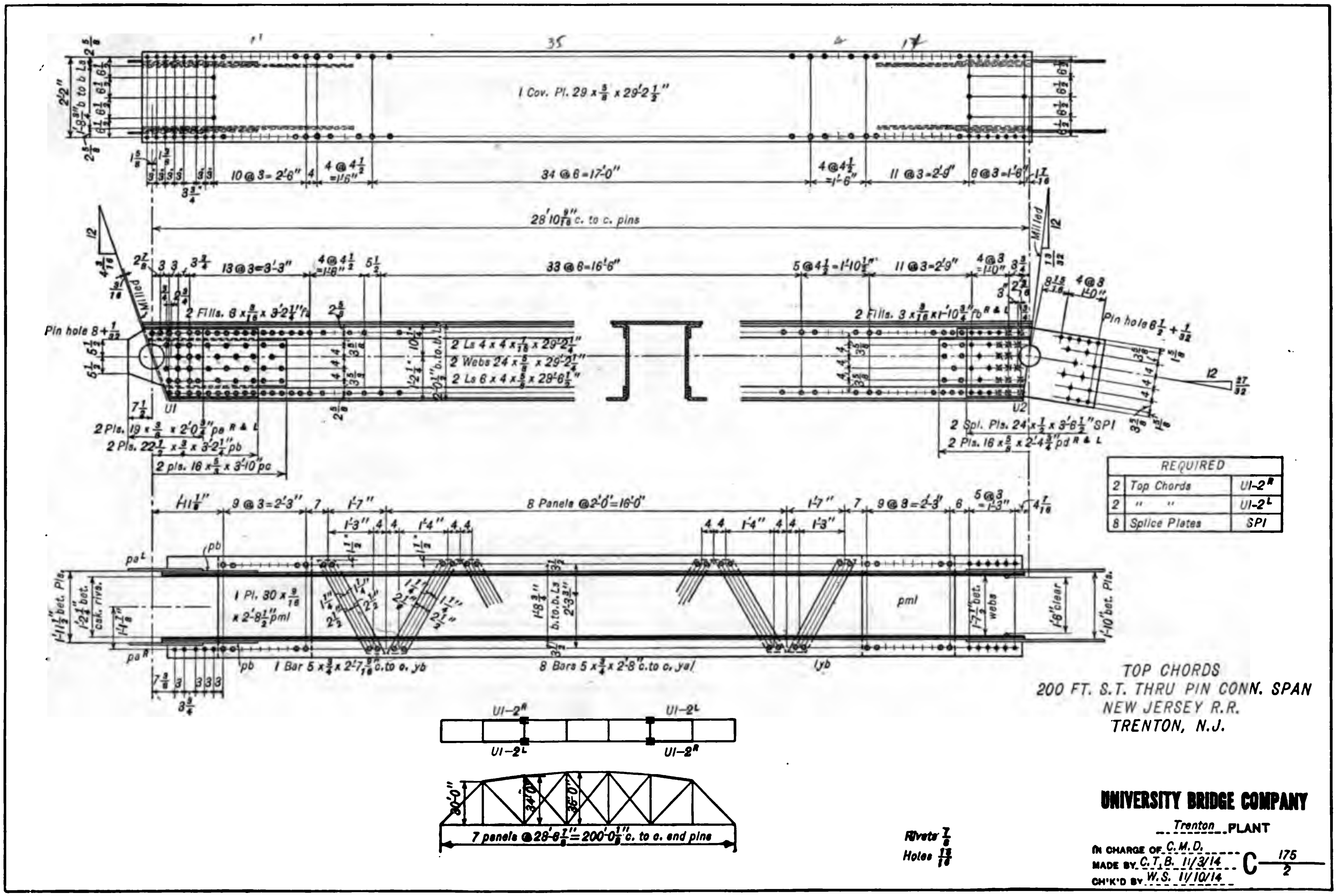
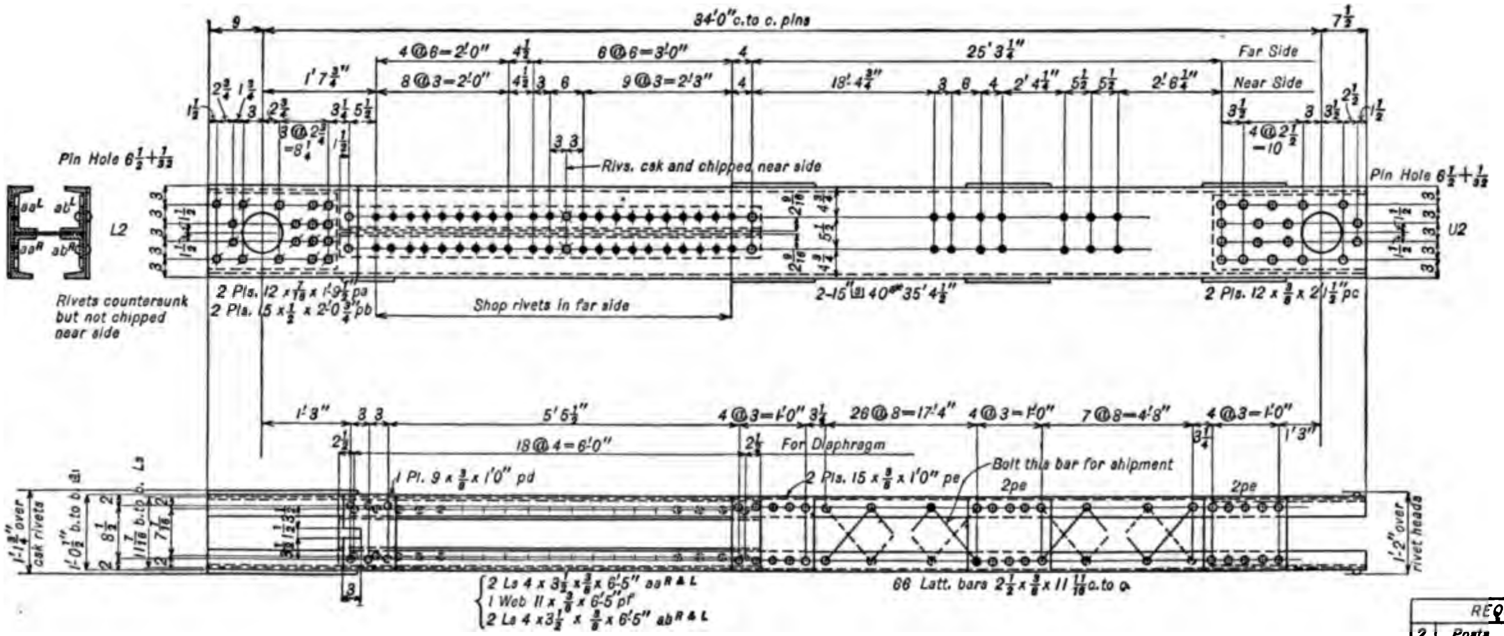


Fig. 128. Top-chord Member for Pin-connected Truss of a Railroad Bridge.

[redacted]



REQUIRED		
2	Posts	L2-U2R
2	Posts	L2-U2L

Note:—Vertical posts are preferably drawn vertically when size of sheet permits.

INTERMEDIATE POSTS
200 FT. S.T. THRU PIN. CONN. SPAN
NEW JERSEY R.R.
TRENTON, N.J.

UNIVERSITY BRIDGE COMPANY
Trenton PLANT

Rivets $\frac{7}{8}$
Holes $\frac{15}{16}$

IN CHARGE OF C.M.D.
MADE BY E.A.S. 11/2/4
CH'K'D BY M.H.D. 11/12/4

C—$\frac{175}{8}$

Fig. 129. Posts for Pin-connected Truss Shown on Preceding Page.

125). Inclined top chords must be spliced at the panel points; this usually necessitates the use of splice plates cut with reëntrant angles (Fig. 128), although these should be avoided if possible on account of difficult shop work (page 76 : 1, IX (4)). For determining the size of splice plates and the number of rivets required, see page 270 : 2.

1. **Reinforcing Plates.** — Channel webs and the web plates of built members are seldom thick enough to transmit the proper stresses to pins. The webs of pin-connected members may be reinforced by auxiliary plates to furnish sufficient bearing area on the pins. When the ends of two compression members bear on opposite sides of a pin, extra plates should be added, or one of the reinforcing plates on each side of each member should be extended, to surround the pin; these plates should not be riveted to the other member (Fig. 127). To avoid interference, these plates may be placed outside of the webs on one member and inside on the other. The purpose of these plates is to hold the members in position during erection and also to keep water out of the joints. For determining the size of reinforcing plates and the number of rivets required, see Chapter XLII, page 284.

2. It is often necessary to use **countersunk or flattened rivets** to prevent the interference of different members during erection. Shopmen are accustomed to look for such rivets in certain usual places so it is sufficient to show them by the proper conventional sign (page 40 : 6). When rivets are countersunk or flattened in unusual places, or when they are so inconspicuous on the drawing that they are likely to be overlooked, a note should be added, as explained on page 40 : 6. Some of the more usual places where the rivets must be countersunk or flattened are: (*a*) in the main plates of the shoes which bear upon the rollers or bed plates; (*b*) in chords, posts, or shoes, to allow for eye-bar heads, for pin nuts, or for the overlapping reinforcing plates mentioned in the preceding paragraph (Fig. 127); (*c*) in reinforcing plates or fillers under splice plates (Fig. 128); (*d*) in the top of the end post under the connection of the portal bracing in order to reduce the number of field rivets (Fig. 127); (*e*) in posts where rivets are required in addition to the field rivets of the floor-beam connections (Fig. 129). Field rivets should be so spaced that they need not be countersunk; thus in Fig. 128, ample clearance is allowed for placing the pin nut in position. The rivets which hold the angles and the reinforcing plates to the web plates are countersunk underneath the splice plate so they can be driven in the shop; note the manner of indicating these rivets by broken lines to show that they are countersunk *back of* the plate instead of *in* the plate.

3. **Protection from the Weather.** — Joints may be partially protected from the weather by splice plates or by reinforcing plates as described in the preceding paragraphs. Joints which are made at the upper panel points may be protected on top by making the connection plates for the top lateral bracing extend entirely across the chords. Care must be taken not to interfere with the free action of a pin-connected joint by riveting these plates to both members. Bottom chord members and joints should be so arranged that no rain pockets are formed. Drain holes should be provided in the larger bracing plates to prevent the accumulation of water (Fig. 125).

4. **Clearance** should be allowed to facilitate the erection of web members between the gusset plates which are attached to the chords. The out to out dimensions of the web members should be made $\frac{1}{8}$ less than the clear distance between the plates.

5. The method of **holding a "counter"** in the proper position on the pin by means of a notched plate is illustrated by plate *pd*, Fig. 129.

CHAPTER XXII

COLUMNS

SYNOPSIS: Specific suggestions with illustrations are given for making working drawings of columns for different types of structures.

1. **Steel columns** form the principal supports of all steel structures other than bridges and similar spans which rest directly upon masonry. They may be of many different forms according to the type of structure. A few typical columns of office-building and mill-building construction have been selected for illustration. Many mill buildings and similar structures have unusual or complicated connections so that it is difficult to proceed with the drawings of the columns until after the drawings of the connecting members have been carried far enough to determine the column connections. When possible it is usually simpler to postpone the column drawings until all connecting members have been drawn. The available time is seldom sufficient to permit this method, however, because the columns are the first members to be erected. The drawings for the columns must logically be among the first drawings sent to the shop, and connections for other members must frequently be provided before the drawings of those other members have been made or even begun. It is often necessary to make a layout of the more unusual connections to determine the necessary dimensions to be used on the column drawings. These layouts may later be used by the draftsmen who make the drawings of the connecting members. Many types of connection recur so frequently that standards are prepared in the drafting rooms of the larger structural companies. These standards simplify the work of the draftsmen and in many cases they simplify the work of the templet makers. In fact, wooden templets for typical connections may often be preserved for future use to save making new ones for subsequent contracts.

2. It is desirable to draw all members in the same relative **position on the sheet** which they are to occupy in the finished structure. Thus, columns are preferably drawn with their longer axes vertical, as in Fig. 137. This is not always feasible on account of the multiplicity of details, for more space is available if they are shown with their longer axes horizontal (i.e., lengthwise). When the columns are drawn horizontally it is customary to show the base or bottom end of the column at the *left* as in Figs. 133 and 135. It is unnecessary to draw the full length of a column to the same scale as the details. The most common scale used for the details is $\frac{3}{4}'' = 1'$. The extreme length of the column is made to suit the available space on the sheet allowing for the necessary views and dimension lines. The details are then placed at approximately proportional distances apart regardless of scale. No breaks in the views need be shown unless the drawing can be made clearer thereby.

3. So many **different types of columns** * are used by different designers it is impossible to show them all. Perhaps the three most common types are the plate and angle, the plate and channel, and the Bethlehem H-section. The drawings for the H-section are comparatively simple since the main section is rolled complete and no continuous lines of rivets are required unless cover plates are used. The connections are similar to those of other columns and it seems unnecessary to illustrate them here.†

* See Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co. Inc., New York.

† For typical connections see the "Catalogue of Bethlehem Steel Shapes," Bethlehem Steel Co., South Bethlehem, Pa.

The plate and channel columns are used for office-building construction. The upper sections are often made of channels with lattice bars, the bars being replaced by cover plates in the lower sections. As the loads increase, the thickness of the plates and the weights of the channels are increased and if necessary the width of the plates and the depth of the channels as well. The plate and angle column is used most extensively for mill-building construction; it is also used for office buildings. Very light sections are made of four angles latticed, but usually solid web plates are used instead of the lattice bars. For very heavy loads larger angles are used with cover plates riveted to their outstanding legs.

1. Typical **plate and channel columns** are shown in Fig. 133. They are detailed for the conditions shown in the diagram of Fig. 158 and in the corresponding column schedule of Fig. 160. The views of the different faces are often lettered for convenience in the identification of the details. The dashed lines of the channel flanges in Faces *A* and *C* need not be drawn full length if the drawing is equally clear without. Since considerable time is required to draw so many long dashed lines it is well to omit portions of them whenever feasible. Column *AB* 1 shows the base angles which connect to the cast-iron bases. At the top, splice plates and angles are provided to connect to the superimposed column section composed of channels of the same depth and cover plates of the same width The lower end of the connecting column would be similar to the lower end of Column *EF* 27, differing only in dimensions since the column is of a different size. The ends of office-building columns are milled so that the loads may be transmitted by direct bearing (page 31 : 1). A more complex splice is required where the depth of the channels and the width of the cover plates change. Such a splice is shown on the column schedule (Fig. 160); it is described more fully on page 276 4. Provision for a similar splice is made at the top of Column *EF* 27. The projecting corners at the top of the splice plates need not be cut (Fig. 277) if they are to be concealed by the fireproofing. For the size of the fillers and the reinforcing plates and the number of the rivets see page 276 : 4. The spacing of the channels and the gages are given on pages 300 and 301. Note that for a given depth of channel the distance out to out of web faces and the distance between rivet lines are constant, while the distance back to back of webs and the gages vary with the weight. This is so arranged in order to standardize the splice plates and the beam connections and also to reduce the number of different lengths of beams.

2. The **important dimension** is the finished length and this is preferably made conspicuous. Since all elevations on the diagrams are referred to the finished floor line it is important to show each floor line on the column drawings and to give the story heights. The beam details are located with reference to these floor lines and the rivets which fasten the cover plates to the channels are then dimensioned to fill in the spaces between the details. Close spacing of four diameters is used at both ends of the column (page 69 : 1 (*d*)) and near the beam connections. Six inch spacing is allowed as a maximum for the remaining distances.

3. Typical symmetrical **beam connections** are shown on Column *EF* 27, and special eccentric beam connections on Column *AB* 1. The whole load from each beam is carried by the seat; the top angle serves to prevent the beam from overturning and to help transmit the wind stresses in the structure. The top angles are left bolted so that they may be removed to facilitate erection. They are also placed $\frac{1}{4}''$ higher than the theoretical top of the beam to provide for the spreading of the flanges of the beam while cooling on the rolls during their manufacture. Note that after the cover plates have been riveted to the channels a box section is formed, the inside of which is inaccessible. In order to provide for the temporary removal and the restoration of the bolts in the top angles of the connections on the channel webs the bolts must pass through the whole column. Unless a similar angle is required directly opposite, a special note must appear to assure the punching of holes in both channel webs for through bolts (face *D*, *AB* 1). The top angles on the cover plate faces are made sufficiently long to permit the use of short bolts through the channel flanges. If shorter angles were used, the necessary through bolts would often interfere with those through the channel webs. The rivets in the channel webs and some of those in the cover plates are inaccessible after the column is assembled; thus, the rivets in the seat angles, stiffening angles, reinforcing plates, and fillers under splice plates, must be driven before the cover plates and channels are bolted together. Special gages are used in the 6″ legs of the seat angles to conform to the spacing of the rivets in the channel flanges. The

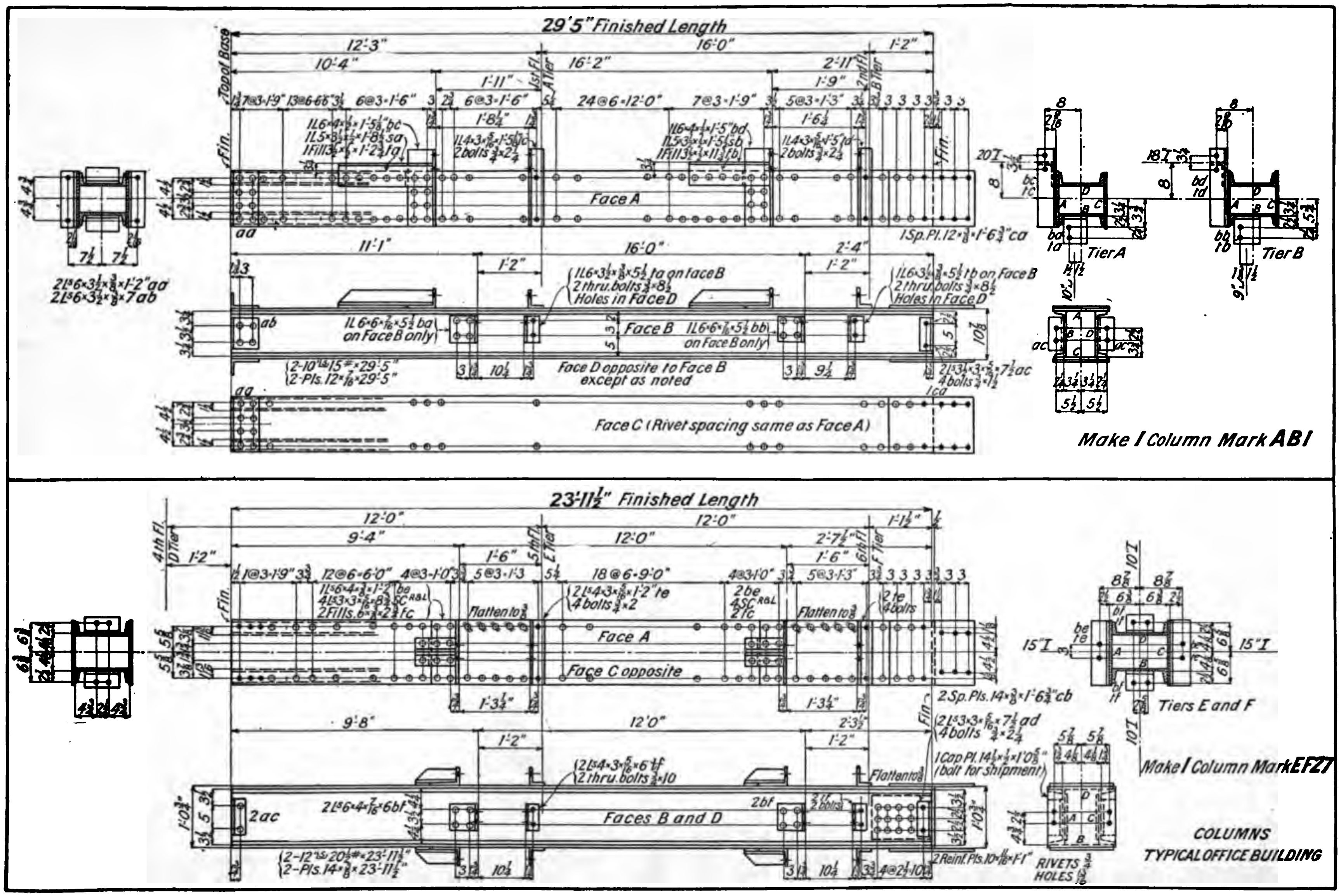


Fig. 133. Typical Office-building Columns.

bottom ends of stiffening angles need not always be cut back as shown. If concealed by the regular fireproofing or by walls or partitions they may be cut square. The draftsman must be familiar with the methods of erection in order to determine which rivets, if any, should be flattened or countersunk to facilitate the insertion of the beams (see Column *EF* 27).

1. **Sectional views** are drawn for each tier of beam connections in order to show the holes in the outstanding legs of the angles. The section is sometimes taken between the top and bottom angles, but more often above the top angles as shown in Fig. 133. In order to draw attention to the fact that the holes should be made in both the seat and the top angles, the holes are often shown in the other views as well (page 40 : 6).

2. Fig. 135 illustrates a **typical mill-building column** drawn for the conditions shown in the plans and diagrams of Figs. 155 and 156. The lower part of the column is made wider than the upper in order that the crane loads may be transmitted to the column base more directly. The width of the lower section is usually made so that the outer face of the cover plate or channel comes directly under the center of the girder. Seat angles and stiffeners are used to provide a suitable girder seat as shown. The girder is secured against overturning by means of a diaphragm. The end stiffeners of the crane girder shown in Fig. 103 are arranged to connect to a diaphragm similar to that shown in Fig. 135. Holes are provided in the channel web for the girder knee brace shown in Fig. 140.

3. **Connections.** — At the top of the column a connection is provided for the roof truss shown in Fig. 116. The web plate of the upper section of the column is connected to the web plate of the lower section by means of splice plates designed to develop the full strength of the upper web (page 276 : 2). The outer angles are continuous, and the shorter angles of the upper section extend downward far enough so that they may be fully developed by rivets which connect them to the lower web. Holes are provided in the outstanding legs of the continuous angles for the girts and struts indicated on the erection diagram (Fig. 156) and detailed in Fig. 147. It is customary to punch these holes in both sides of the end columns the same as in the intermediate columns, although the side girts do not extend past the center lines. These extra holes may not all be used, although as far as possible the girts on the ends of the building are arranged to connect to them. The cost of punching a few unused holes does not greatly exceed the cost of omitting them because of the time required to make and to follow special notes, and furthermore, the wrong holes might be omitted. In case the building is later extended the extra holes may be used. The bottom strut at the end of the building (*S* 7 Fig. 140) connects to the flange of the channel of Column *C* 1 instead of to the outer angles to make a more rigid connection and to reduce the lengths of the struts and the diagonal sway bracing. The sway bracing connects to the top of Column C 1 on the inner face by means of a bracket (*D* 13, Fig. 140).

4. A typical **column base** as described on page 290 : 1 is illustrated in Fig. 135. An excessive number of countersunk rivets in the base plate and the cap plate should be avoided. It is usually impractical to swing a long column around so that these few rivets can be driven by machine and consequently they must be driven by hand riveters. Since these rivets do not have a very important function their number should be reduced to the minimum. Holes for anchor bolts are made $\frac{5}{16}''$ or $\frac{3}{8}''$ larger than the diameter of the bolts to facilitate placing the columns in position when the bolts are set first, or drilling holes in the masonry for the bolts after the steel is in position.

5. **Milling.** — A column which supports crane runway girders is invariably milled at the base and at the crane seat in order that the crane loads may be transmitted largely by direct bearing. The web plate is not always milled at the seat along with the angles and channel because of practical difficulties.

6. Most of the **vertical dimensions** to the bottom of the column extend only to the upper face of the base plate instead of to the extreme bottom. In this way the dimensions best serve the templet maker and the inspectors who use them before the base plate is in place. In some companies, however, the dimensions extend to the extreme bottoms of the column bases. Draftsmen must be particularly careful to give due consideration to those dimensions which may be affected by the thickness of the base plate. When many different connections appear on a single face of the column, as in the outer face of *C* 1 and *C* 2, "extension figures" should be given from each connection to the base plate, in addition to the dimensions from one connection to the next. The extension figures are

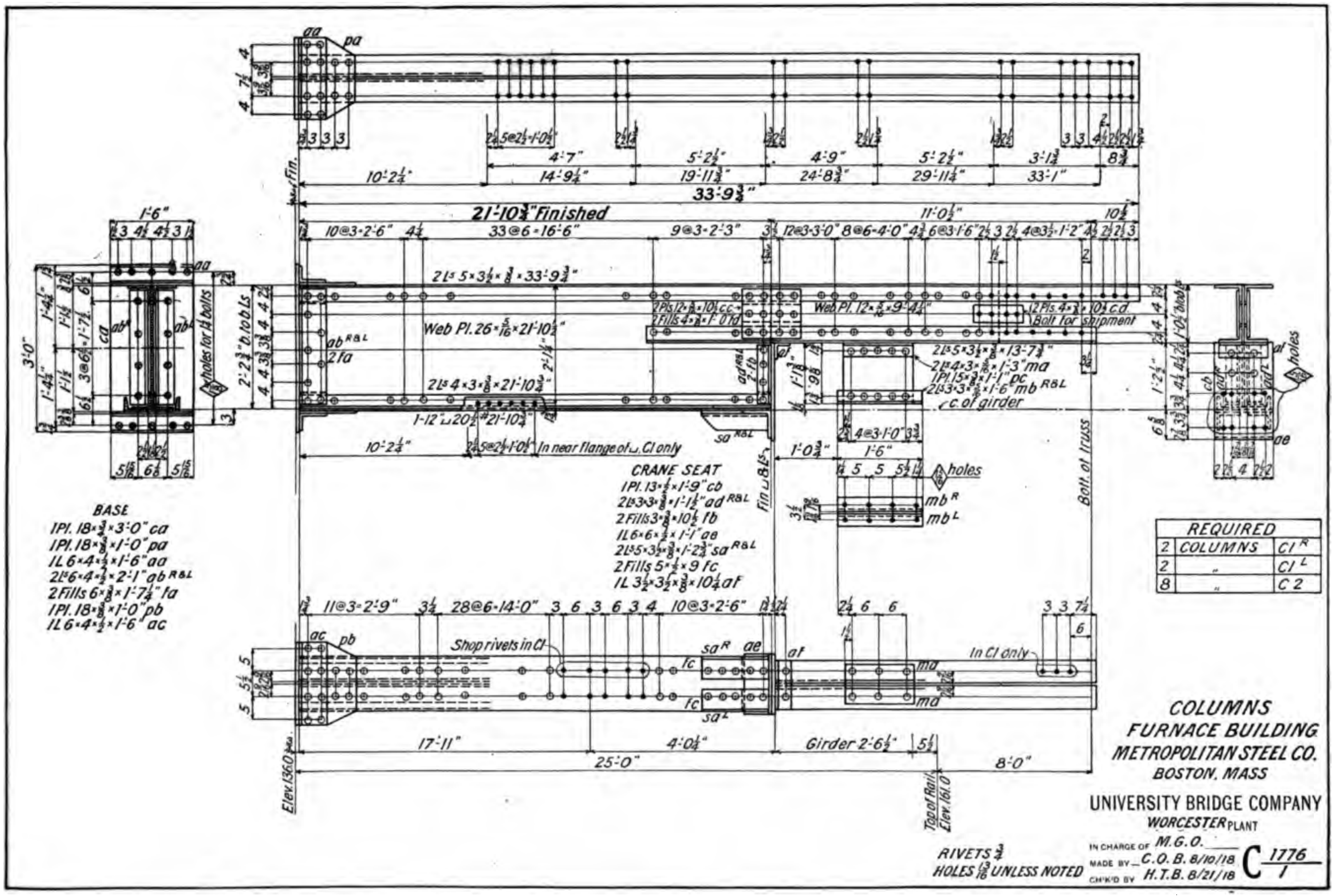


Fig. 135. Typical Crane Column for a Mill Building.

convenient for the templet makers and the inspectors so that they can locate all the connections with a single setting of the tape. The figures are of convenience also to the draftsmen and checkers when referring to a drawing to obtain differences in elevation between connections. When making the drawing the draftsman should record the extension figures for each connection as soon as that connection is detailed. The dimension should extend preferably to the point dimensioned on the diagram, as for example, the back of a girt or a strut angle. Similar connections may then be detailed before other types of connection are designed, and later the other types may be inserted in the proper places. After all connections are located the distance from one connection to the next may be found simply by subtracting the corresponding extension figures. Some companies require a complete line of dimensions from center to center of holes, but this seems unnecessary, particularly when there are comparatively few holes in each group. If the groups are located as shown in Fig. 135 the dimensions can often be taken directly from the diagram, and if the shopmen are forced to use these dimensions instead of calculated distances between adjacent holes one source of error is eliminated.

1. **Standard gages** in column angles are not used in all cases. In the outstanding legs of the main angles and of the angles in the diaphragm and similar places it is desirable to make the distances from center to center of holes a multiple of $\frac{1}{4}''$ or preferably $\frac{1}{2}''$. Thus the use of sixteenths and eighths in the main dimensions of connecting members is avoided. The gages in the angles may result in sixteenths or eighths, but the use of small fractions is confined to relatively few dimensions. Similarly, if girts or other members connect to column angles of different sizes, more members may be made alike if the gages in the column angles are made the same. For example, the gages in 5 and 6 inch legs may be made equal, and those in $3\frac{1}{2}$ and 4 inch legs also. A single rivet line is usually used in each leg, even if 5 or 6 inches.

2. Two other types of mill-building columns are shown in Fig. 137. Column C 3 is a column for the gable end of the mill building illustrated in Fig. 156. The end truss is made similar to the intermediate trusses and is designed to span the full width of the building without intermediate supports. The chief function of the **gable column** is to support the end framing (girts, struts, etc.) below the bottom of the truss. The upper end of the column is supported laterally by being connected to the roof truss at a point where the truss is braced in the plane of the bottom chord; but in order to avoid stresses for which the truss is not designed, the connection angles at the top of the column are provided with vertical slots to permit the free deflection of the truss. The base is designed to transmit the whole load through rivets so that it is unnecessary to mill the column. In general, a column is not milled unless the load is more than about 40,000 or unless the column is to support a crane or other moving load. Holes are provided in the outstanding legs of the column angles for girts and struts as in the columns of Fig. 135.

3. Columns C 4 and C 5 are light **latticed columns** with provision for a roof truss connection at the top. An eccentric beam connection is inserted between the two groups of lattice bars. Connections for light lean-to rafters and for girts are shown in the outer faces. Separate views of these faces are drawn for the two columns in order to show the differences clearly. When lattice bars are placed *between* the angles as in this type of column the distance between angles is not constant; the distance may be determined by the thickness of the plates as *pa* or *pb*, by the thickness of a single lattice bar, or by the thickness of two overlapping bars. Consequently either the gages in the angles or the distance between holes must vary. Usually it is desirable to maintain a constant distance between holes so that like members may be connected at different parts of the column. Rather than to dimension each group of holes separately it is well to omit the gages altogether and to let the templet maker make proper provision for the variation; then the distance between holes need be dimensioned only once for each view. For the size of lattice bars and batten plates see page 216:2–3. No tie or batten plate is needed at the top because the heel plate of the roof truss will serve the purpose after it is in position.

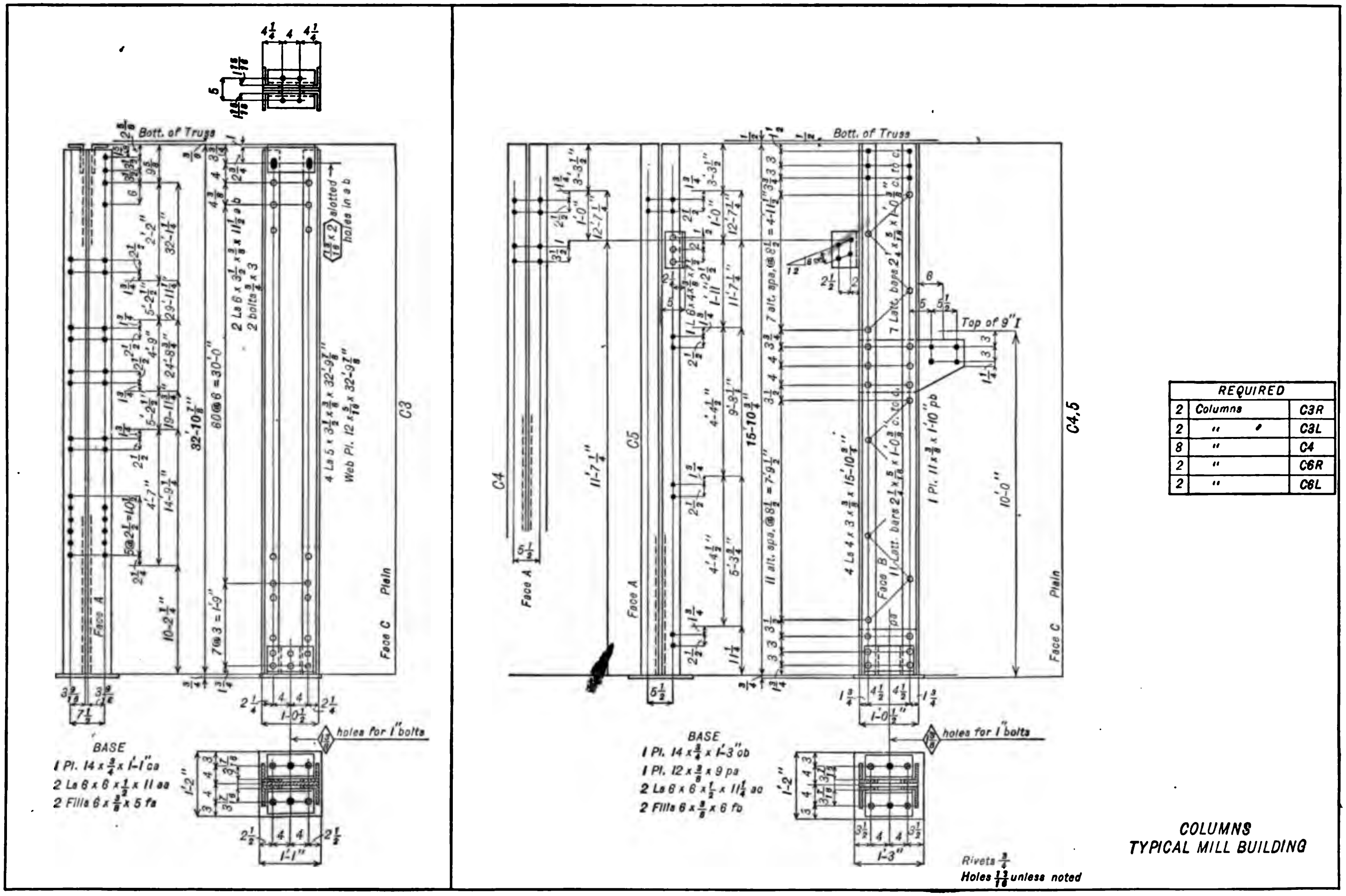


Fig. 137. Typical Light Columns for a Mill Building.

CHAPTER XXIII

BRACING SYSTEMS

SYNOPSIS: A discussion of the types of bracing used under different conditions, with illustrations.

1. Some system of **bracing** is usually required to secure a structure against forces which tend to distort or overturn it. These forces may result from the wind, from moving loads, or, during erection, from derricks or travelers. Diagonal bracing is the most effective but it cannot be used where it would interfere with the use of the structure, as for example across doorways. When it is not feasible to place diagonals entirely across the panel to be braced, special brackets, knee braces, or portal struts may be employed. In some structures the riveted joints may give ample security so that no special bracing is required. In other structures only temporary bracing is required during erection, as for example the bracing between steel columns which are later to be imbedded in solid masonry walls.

2. Bracing systems with full diagonals may be **considered as trusses** and so designed. The chord stresses of these trusses are taken by the members to which the bracing connects, as for example the columns, the girders, or members of other trusses. The "posts" or transverse compression members may be either special struts or else members which are already provided, such as floor beams, cross frames, or purlins. The diagonal stresses in any panel may be resisted by a single member placed along either diagonal of the panel; if in one position it will be in tension but if in the other position it will be in compression. Usually these diagonal members are designed for tension and are placed accordingly. To provide for forces which might cause a reversal of stress, members may be p'aced along both diagonals to form "cross bracing;" only the diagonal which is in tension is considered to act at any one time. The sizes of the members and the number of rivets are often standardized for similar conditions in order to simplify the design.

3. Bracing systems with full diagonals should be **statically complete**; that is, diagonals should not be used unless they are supplemented by the proper struts. Special struts are not always required; for instance an eave strut of a mill building may serve as a girt, as a purlin, and also as the end strut of the bracing in three different planes, viz: the vertical sway bracing between columns, the horizontal bracing between the bottom chords of the trusses, and the bracing parallel to the plane of the roof between the top chords of the trusses.

4. **The lines of stress** of all members which are connected by a single gusset plate should meet approximately in a common point to minimize the secondary stresses. Adherence to this rule is seldom strictly enforced, however, because a slight deviation will permit the use of an auxiliary system of working points, as explained on page 108 : 5. A plate which connects only a single diagonal to another member should be so arranged that the line of action (rivet line) of the diagonal will fall within the group of rivets which connect the plate to the other member, in order to reduce the eccentricity. See Fig. 140.

5. **Arrangement.** — The drawings for cross bracing are usually so made that the diagonals are shown in the proper relation to each other and to the members to which they connect. The system of working lines can then be easily checked, and the connections to other members may be readily compared with the drawings of the corresponding members. The centers of the end holes in the diagonals are usually chosen

as working points. The system of working lines may be plotted to a smaller scale than the details in order to save space; some of the simple diagonals or struts may be shown separately for the same reason. In the simplest form of cross bracing the diagonals are so turned that they may pass each other without interference, as shown in the cross frames, Fig. 142. More frequently the outstanding legs of the two angles are made to face the same way even though one angle has to be cut and spliced at the intersection. In this position they occupy less space and they are less liable to interfere with other members; it is often necessary to turn them this way to obtain the desired clear opening without increasing the height or width of a structure.

1. **Initial Tension.** — Diagonal bracing must be tight in order to be most effective. A long diagonal will sag under its own weight during erection unless it is drawn tight before it is bolted or riveted. Considerable racking of the structure could take place without removing this sag or stressing the member. In order to make a structure more rigid by causing the diagonals to act at once, the length from center to center of the end holes is made less than the calculated distance. The member may then be drawn into position for bolting or riveting by driving a tapered drift pin into the holes. Since the holes are punched $\frac{1}{16}''$ larger than bolts or rivets the member should be shortened $\frac{1}{16}''$ to take up the "play" in the holes and at least another $\frac{1}{16}''$ to overcome inaccuracies in punching and other factors. Tightness is thus insured even though a certain amount of initial tension may result. The total amount to be deducted from the calculated distance from center to center of end holes should be either $\frac{1}{8}''$ or $\frac{3}{16}''$, whichever will make the main dimension a multiple of $\frac{1}{8}''$. In this way the half lengths will be expressed in multiples of $\frac{1}{16}''$, and 32nds will be avoided. Sometimes the amount deducted is noted, as in *T* 1, Fig. 143. The chief benefit of such a note is to give assurance that provision has been made for some deduction. No deduction should be made for comparatively short stiff members such as the diagonals in the cross frames or the lateral bracing between the girders of a deck railroad bridge, because it would be difficult to connect them.

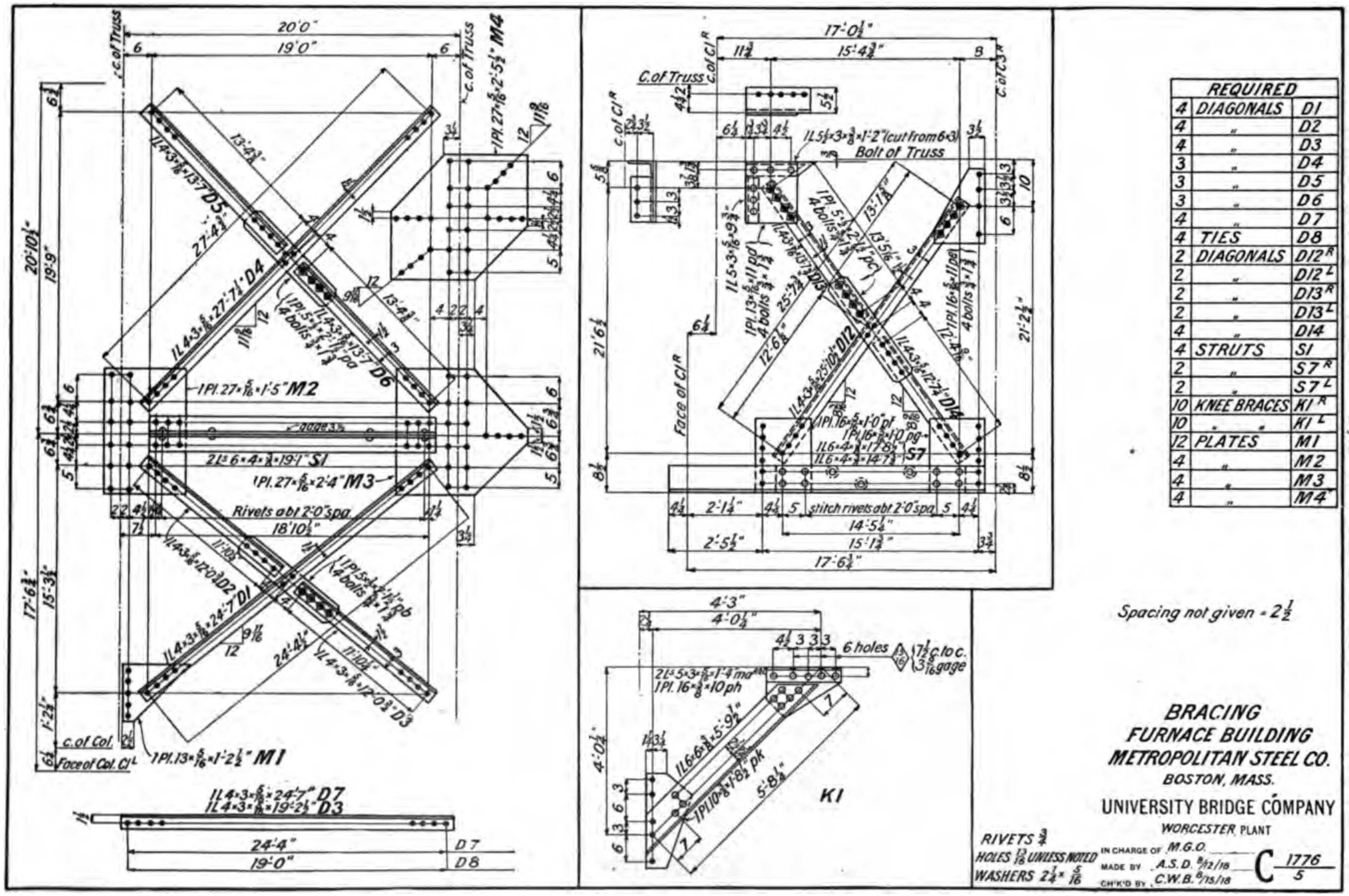
2. **Connections.** — The diagonals and the connection plates are usually shipped separately although some of the smaller plates may be fastened to the angles. The bracing for all bridges and many buildings are fully riveted in the field. The bracing for parts of some buildings may be bolted if the specifications will permit. When the field connections are to be bolted, similar shop connections may be bolted also (Fig. 140); if there are other shop rivets to be driven in the same member it is about as cheap to use rivets instead of shop bolts in the end connections.

3. Special **gages** are often used in bracing angles. The rivet line of a single angle is used as the working line. If this working line is placed in the center of the leg the connections may be detailed to better advantage. The clearances on opposite sides are thus made more nearly equal regardless of which way the angle is turned when erected. In angles with legs less than 3″ standard gages should be used to allow greater driving clearance for the rivets or bolts.

4. Typical **illustrations** have been selected to show the common forms of bracing used in different structures. The general arrangements are shown in the erection diagrams of Chapter XXV (page 151), but the details are shown in the drawings of this chapter.

5. **A mill building** can have no system of bracing which obstructs the interior and prevents the free movement of cranes or other objects. Cross bracing is commonly used in the sides, the ends, and the roof, while knee braces are used to stiffen the connections of the intermediate trusses to the columns. Angles are used as diagonals in the plane of the bottom chords of the roof trusses, and in the vertical sway bracing between columns, both on the sides and on the ends. Rods are used as diagonals in the planes of the top chords of the trusses, and in the sides and the tops of the monitors. The end panels of the building are usually fully braced in all these planes. See Fig. 156. Only every third or fourth intermediate panel is similarly braced with diagonals although the struts extend the full length of the building. The struts in the braced panels are usually heavier than those in the unbraced panels. In the plane of the bottom chords additional diagonals are so placed as to form a large system of cross bracing which extends the full width of the building. Vertical bracing is sometimes used between trusses at the center.

6. Fig. 140 shows the **bottom chord bracing** and the end sway bracing for the mill building represented in Fig. 156. The working lines are



REQUIRED		
4	DIAGONALS	D1
4	"	D2
4	"	D3
3	"	D4
3	"	D5
3	"	D6
4	"	D7
4	TIES	D8
2	DIAGONALS	D12 R
2	"	D12 L
2	"	D13 R
2	"	D13 L
4	"	D14
4	STRUTS	S1
2	"	S7 R
2	"	S7 L
10	KNEE BRACES	K1 R
10	" "	K1 L
12	PLATES	M1
4	"	M2
4	"	M3
4	"	M4

Fig. 140. Typical Bracing for a Mill Building.

referred to the working lines of the trusses and columns for convenience in checking the field connections. The working lines through the end holes of the diagonals are placed to make the clearances on each side approximately equal, as explained on page 77 : 1. The main dimensions of the diagonals between end holes are made less than the calculated distances, as explained on page 139 : 1. For the sake of appearance, the corners of all plates should be concealed by the angles. They should preferably be made to fall at the edges of *single* angles, but this is not of sufficient importance to justify any increase in the size of the plate. The corners of diagonal cuts can be shown at the edges of angles drawn in position, or dimensions can be given if the angles are not in place. On account of the difficulty of holding narrow plates in the shears while long diagonals are being cut, the corners of splice plates such as *pa* or *pb* are often cut at 45° instead; these cuts are shown but not noted.

1. **Unsymmetrical bracing** is illustrated by the end sway bracing of Fig. 140. The bottom strut *S* 7 connects to the flange of the channel of *C* 1 (Fig. 135). The roof truss serves as the top strut, the bracket at the top of *D* 13 being connected both to the truss and the column; note that the angle which connects to the truss is cut to clear the fillet of the angle *ma* (Fig. 116). After the working points have been located, the length and the slope of each diagonal can be found in the usual manner. The position of the intersection of the diagonals relative to the lower working points can be found by solving the triangle of which these three points are the vertices. The angles may be easily determined and the horizontal side is known. The remaining sides may be found by equating the ratios of the sides to the sines of the opposite angles. For example, the angles are determined from their cotangents as follows:

$$\begin{aligned} \log 14'\,5\tfrac{1}{4}'' &= 1.15949 \\ \log 21'\,2\tfrac{1}{2}'' &= \underline{1.32651} \\ \log \cot\ &= 9.83298 \\ \text{angle} &= 55^\circ\,45' \end{aligned} \qquad \begin{aligned} \log 15'\,4\tfrac{3}{4}'' &= 1.18740 \\ \log 21'\,6\tfrac{1}{2}'' &= \underline{1.33328} \\ \log \cot\ &= 9.85412 \\ \text{angle} &= 54^\circ\,27' \end{aligned}$$

$$\text{third angle} = 69^\circ\,48' = 180^\circ - 55^\circ\,45' - 54^\circ\,27'$$

The remaining sides are found as follows:

$$\begin{aligned} \log 14'\,5\tfrac{1}{4}'' &= 1.15949 \\ \log \sin 55^\circ\,45' &= 9.91729 \\ \text{colog} \sin 69^\circ\,48' &= \underline{0.02757} \\ &\ 1.10435 \\ \text{length} &= 12'\,8\tfrac{5}{8}'' \end{aligned} \qquad \begin{aligned} \log 14'\,5\tfrac{1}{4}'' &= 1.15949 \\ \log \sin 54^\circ\,27' &= 9.91041 \\ \text{colog} \sin 69^\circ\,48' &= \underline{0.02757} \\ &\ 1.09747 \\ \text{length} &= 12'\,6\tfrac{3}{16}'' \end{aligned}$$

These lengths should be reduced by $\frac{1}{16}''$ because the total lengths have been shortened $\frac{1}{8}''$ to insure tightness (page 139 : 1).

2. *K* 1 is a typical **knee brace** to connect a crane girder to a column. Two types of connection are shown, one at the top and the other at the bottom; note that in each type the line of action falls well within the group of rivets (page 138 : 4).

3. Typical **bottom lateral bracing** for a through girder bridge is shown in Fig. 142. The plates are connected both to the girders and to the floor beams. Small angles *ma* are used to connect the diagonals to the bottom flange angles of the stringers; this is done to prevent longitudinal movement of the stringers due to traction and braking stresses. The bottom lateral bracing of a truss bridge is similar to that of the girder bridge except that it is usually made heavier because of the increased span. The single angles are often replaced by double angles.

4. **The top lateral system** of a through truss bridge cannot be made a complete system of cross bracing extending down the inclined end posts to the supports because a clear passageway must be maintained at the ends of the bridge. Cross bracing is used in each panel of the main top chord, but a portal strut is used to transmit the corresponding stresses to the end posts which act as girders in carrying these stresses to the supports. The portal struts and the intermediate struts or sway braces are made as deep as the required clearance will allow; in general they are made much heavier than they were a few years ago, to give greater rigidity, particularly in railway bridges. The intermediate sway braces are usually made of four angles latticed, as *SB* 1, Fig. 143; more elaborate bracing is used in bridges with inclined top chords because of the greater depth available. Many different types of portal struts are used, as for example the solid web type, *PS* 1, Fig. 143, or the latticed type, Fig. 149. Since the portal strut is in an inclined plane, the outer angle at the bottom forms a trough which should be provided with

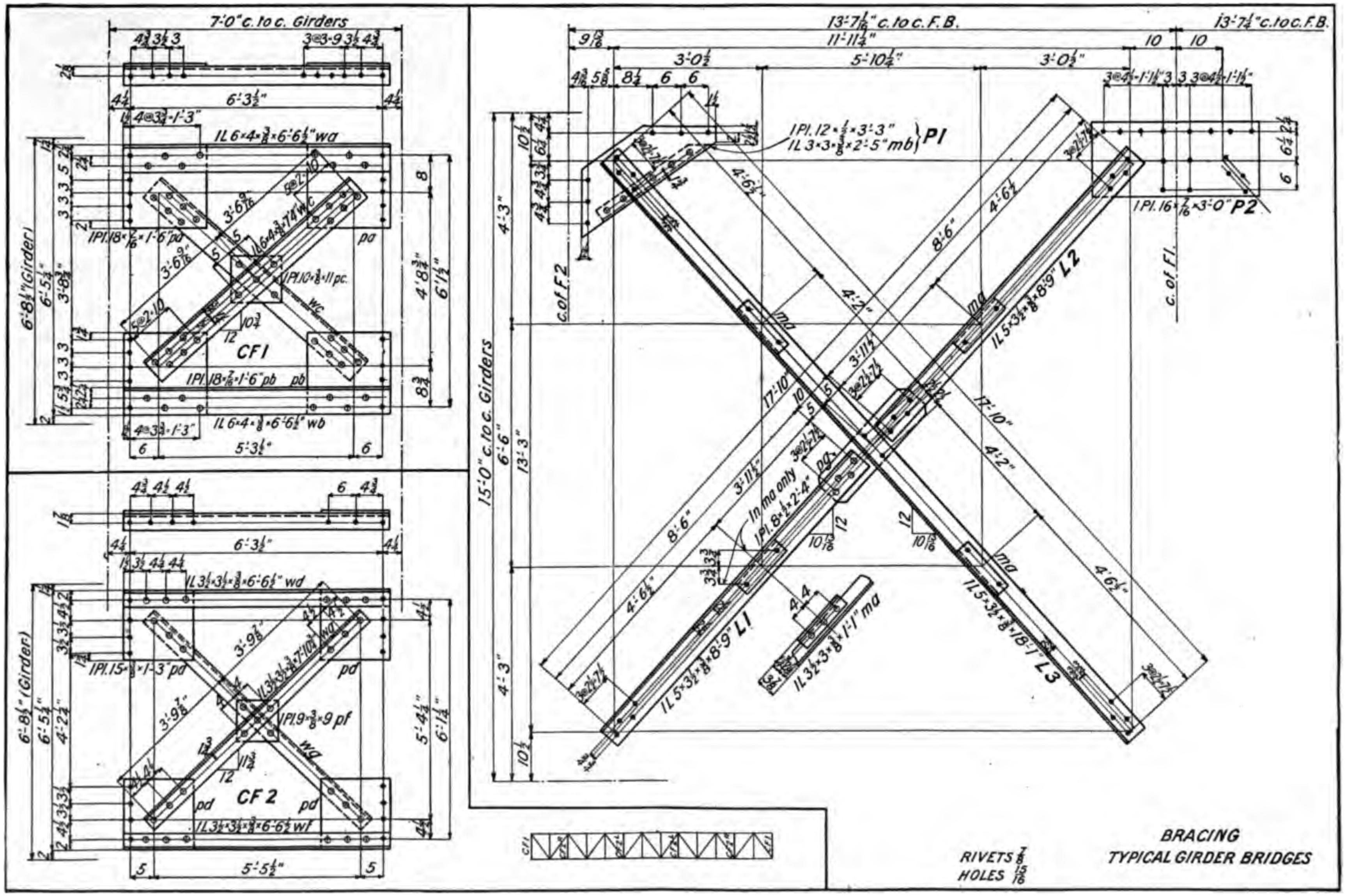


Fig. 142. Typical Bracing for a Girder Bridge.

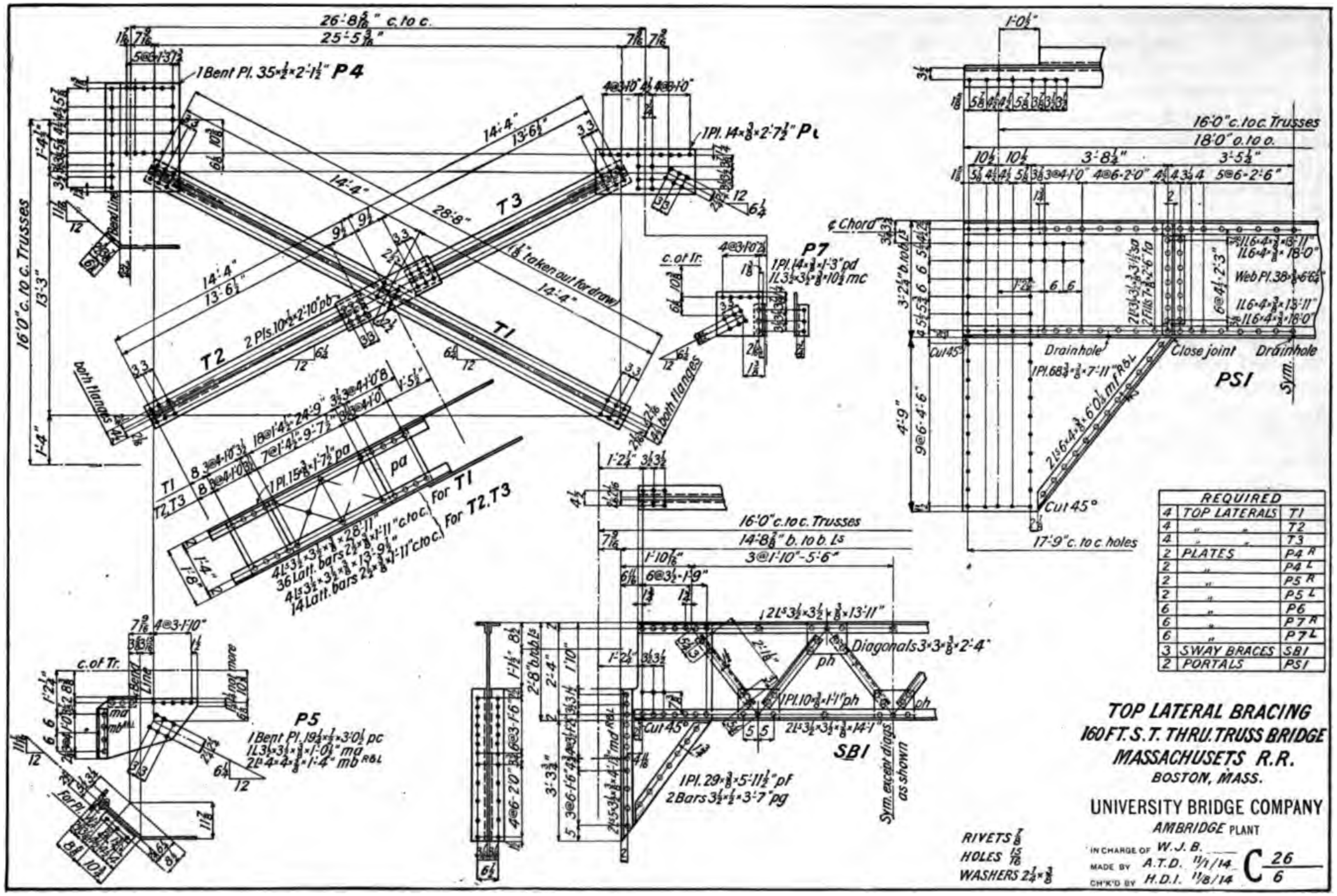


Fig. 143. Typical Top Lateral Bracing for a Railroad Truss Bridge.

drain holes to prevent the accumulation of water. The corners of some of the angles of a portal may be cut at 45° if the appearance is improved thereby. The intermediate top struts or sway braces can usually be made deeper than the top chords; lattice angles are used when the depth is too great for the use of lattice bars. The connection plates at the ends (*pf*) must be cut out to clear the angles of the top chords; this is one of the few places where reëntrant cuts are used. Such cuts are usually made by punching a series of connected holes the same size as the rivet holes, and then chipping off the remaining points with a pneumatic chisel. A small curve is drawn at the vertex of the reëntrant angle to show that it is unnecessary to chip out the extreme corner to form a right angle. The top laterals or diagonals are composed of one or two angles in the lighter bridges and two or four angles latticed in the heavier bridges. The latticed members connect to plates which are fastened to the tops and the bottoms of the chords; the depth is thus determined by the depth of the chord members, being made $\frac{1}{16}''$ or $\frac{1}{8}''$ less than the clear distance between the lateral plates. The upper lateral plates which serve to cover joints in the chords extend the full width of the chords; other plates connect to the inner side only. Plate *P* 6 connects to the chord *U* 1-3 (Fig. 124), and to the top of *SB* 1. *P* 7 connects to the upper face of the angle on the bottom of the chord and to the vertical plate of *SB* 1. The diagonals fit between these plates. Plate *P* 4 connects to the top of the chord and it is bent up to connect to the top angle of the portal strut *PS* 1. The holes in the bent up portion should appear as ellipses instead of circles, but on account of the difficulty in drawing small ellipses, circles are sometimes used when no misunderstanding is likely to result. *P* 5 connects to the bottom angle of the top chord, to the vertical face of the end post, and to the end plate of the portal strut *PS* 1. The angles on this bent plate are not shown in true projection because the drawing would be made unnecessarily complicated. The bend in the plate is clearly shown and dimensioned. If the holes in the angles *mb* were to be shown accurately as circles two additional views would be required. It seems equally clear and more convenient to draw the top view of the angles directly above the elevation and to show them conventionally by three lines to facilitate dimensioning the rivets and holes; confusion would result if both the rivets and the holes were shown in this view in exact orthographic projection. Care should be taken, however, to give dimensions only in the views where the corresponding distances are shown in true projection.

1. **Brackets.** — The compression chords of bridges where no top laterals can be used must be braced in some other way to give intermediate support to the compression members to prevent them from buckling. "Pony trusses" are trusses which are not deep enough to permit the use of overhead bracing. They may be braced by means of brackets such as that shown in Fig. 144. Similarly, the compression flanges of through plate girder railroad bridges are braced transversely by means of brackets or deep gusset plates at the connections of the floor beams to the girders, as shown in Fig. 99. Through plate-girder highway bridges are braced in much the same manner except that it is impractical to use as wide plates on account of the encroachment upon the clear roadway. The narrower plates are sometimes supplemented by other plates or brackets on the outside of the girders.

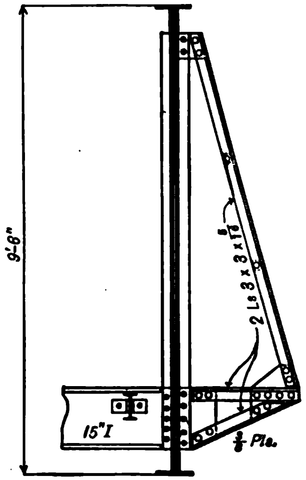


Fig. 144.

2. Deck plate girder bridges are braced vertically by means of **cross frames** such as shown in Fig. 142. The frames at the ends of the bridges (*CF* 1) are made heavier than the intermediate ones (*CF* 2). The cross frames are connected to the stiffening angles of the girders and also to the plates of the lateral bracing which are attached to the under sides of the top flanges. The key plan in Fig. 142 shows a typical layout of the lateral bracing, single angles being used for the diagonals and also for the struts between the cross frames. This lateral bracing is often drawn on the sheet with the girders so that the connection plates can be shown in position, in order to save the duplication of many of the dimensions. The spacing of the holes in the cross frames and the stiffening angles should be so arranged that a clearance of $\frac{1}{16}''$ is allowed

between the tops of the cross frames and the lateral plates; in case the cross frames connect to bottom lateral plates as well, $\frac{1}{16}''$ clearance is allowed at the bottom and $\frac{1}{8}''$ at the top.

1. The wind bracing of an **office building** depends upon so many factors that it is impossible to discuss it comprehensively here.* The form of the building, the number of floors, the number and the position of the partitions, and the position of the doors and windows, are points to be considered in selecting the form of bracing. Diagonal bracing, portal bracing, knee bracing, and brackets are the more common types. Typical brackets for connecting office-building beams and girders to columns are illustrated in Fig. 145.

* See Fleming's "Six Monographs on Wind Stresses," Kirkham's "Structural Engineering," or Ketchum's "Structural Engineers' Handbook," all published by McGraw-Hill Book Co. Inc., New York; also Burt's "Steel Construction," American Technical Society, Chicago.

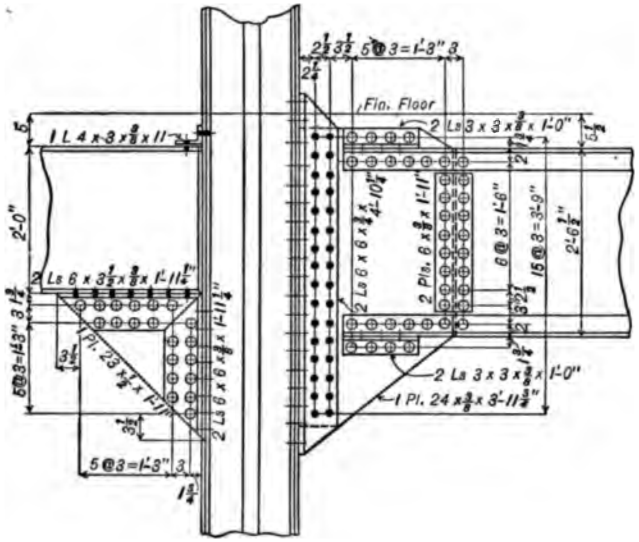


Fig. 145.

CHAPTER XXIV

MISCELLANEOUS FRAMING

SYNOPSIS: Girts, struts, plate work, and skew work.

1. **Illustrative Drawings.** — In this chapter are given a few miscellaneous drawings to supplement those of the preceding chapters. Taken as a whole, the drawings in this book have been chosen to illustrate different types of members and different methods of detailing. No attempt has been made to show the working drawings of complete structures, although in some cases several different members of the same structure are shown. It is felt that there are enough drawings to illustrate the fundamentals of structural drafting, and at the same time to show many of the more common kinds of connection.

2. In Fig. 147 are shown some of the **girts** and struts of the mill building of Fig. 156. *F* 7 and *F* 8 are typical intermediate side and end girts, with holes in the outstanding legs for bolting the window frames in position. The side girts in the end panels are the same as in the intermediate panels, but the end girts which connect to the corner columns are made long enough to support both the side and the end corrugated steel at the corners. Thus *F* 9 is an end girt which connects to the inside face of the outer angles of column *C* 1 (Fig. 135), and extends beyond the column until it is flush with the outer edge of the side girts. If possible the connections for these end girts are made so that the holes in the corner columns are spaced the same as in the intermediate columns; this not only simplifies the drafting and the shop work but it also facilitates future extension of the building. *F* 10 is an intermediate support for the louvres; it connects to the purlins which are attached to the side of the monitor (Fig. 117) as indicated in the side elevation, Fig. 156. *M* 5 is a hanger or sag bar which is used in place of a sag rod to support a long girt wherever a window prevents the use of a rod.

3. *S* 2 and *S* 3 are typical eave **struts**; they serve as **girts** and as purlins to support the corrugated steel, as well as struts for three different systems of bracing (page 138 : 3). Each channel is stiffened by an angle which extends the full length between the connection plates. Holes are provided in *S* 2 for the sway bracing which, at the bottom, connects to the strut *S* 4 shown below. *S* 4, *S* 5 and *S* 6 are typical two-angle struts used in the sides and the ends of the building; they serve also as girts.

4. Fig. 148 shows a drawing of some of the steel plates of a roof for a cast house around a blast furnace. This is not as common an application of **plate work** as floor plates or the plates in a tank, but in simple form it illustrates the method of dimensioning. In tank work the rivets must be placed closer together and the outer edges of the overlapping plates must be beveled for calking, in order to make the joints watertight (page 69 : 1).

5. **Skew Work.** — Some of the connections encountered in structural work require more than ordinary computation in determining the proper angles and dimensions. Common examples of this class of work are hip and valley roof construction, skew portal bracing, bins, chutes, hoppers, etc. The details and the corresponding calculation for the construction of different types of intersecting roofs have been so fully treated by the author in another volume * that they are not even summarized here. A complete mastery of that book should enable the draftsman to apply the principles to the solution of other problems of a similar nature. One form of skew portal is illustrated in Fig. 149; it is designed to connect to

* "Hip and Valley Rafters," John Wiley and Sons, Inc., New York.

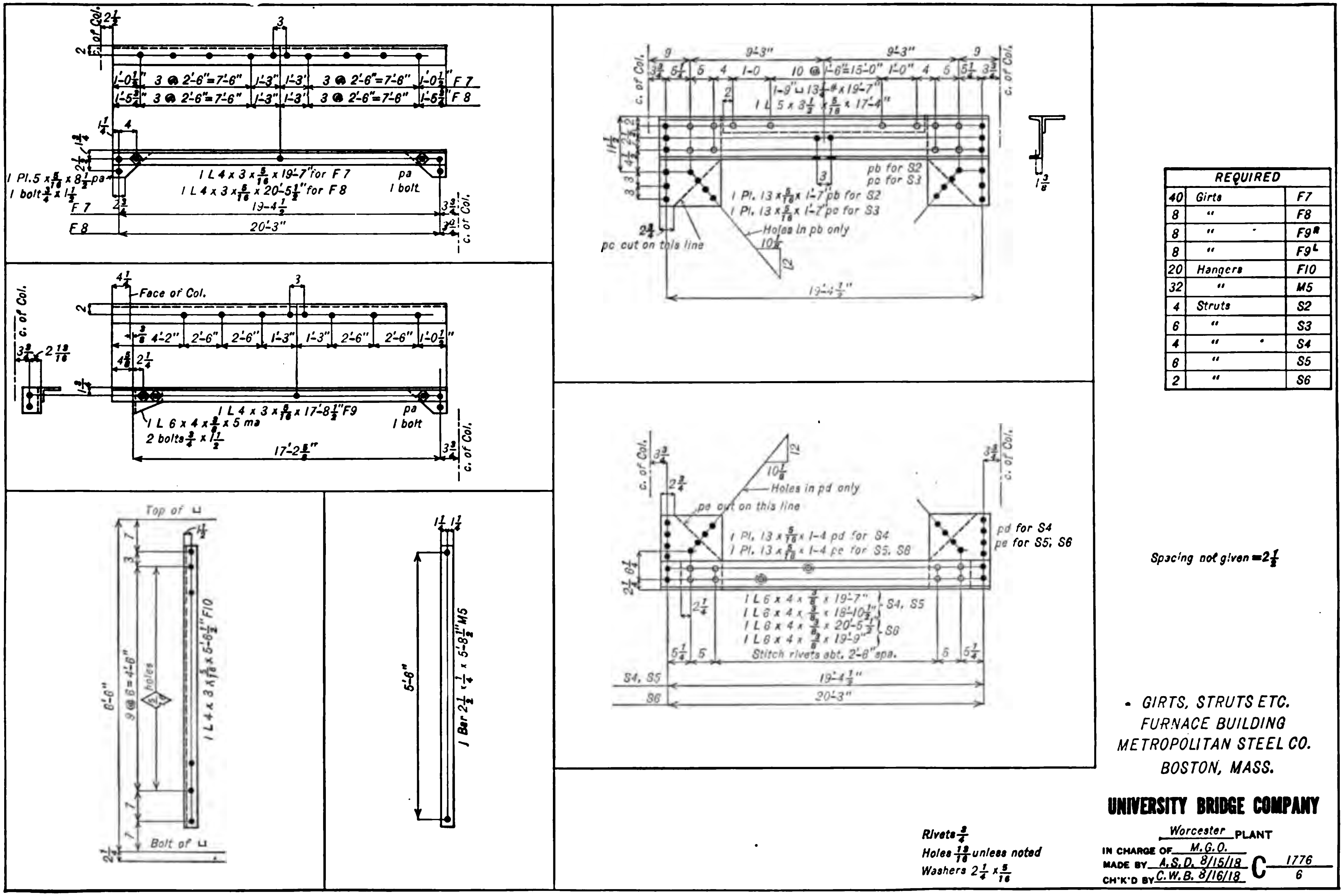


Fig. 147. Typical Girts and Struts for a Mill Building.

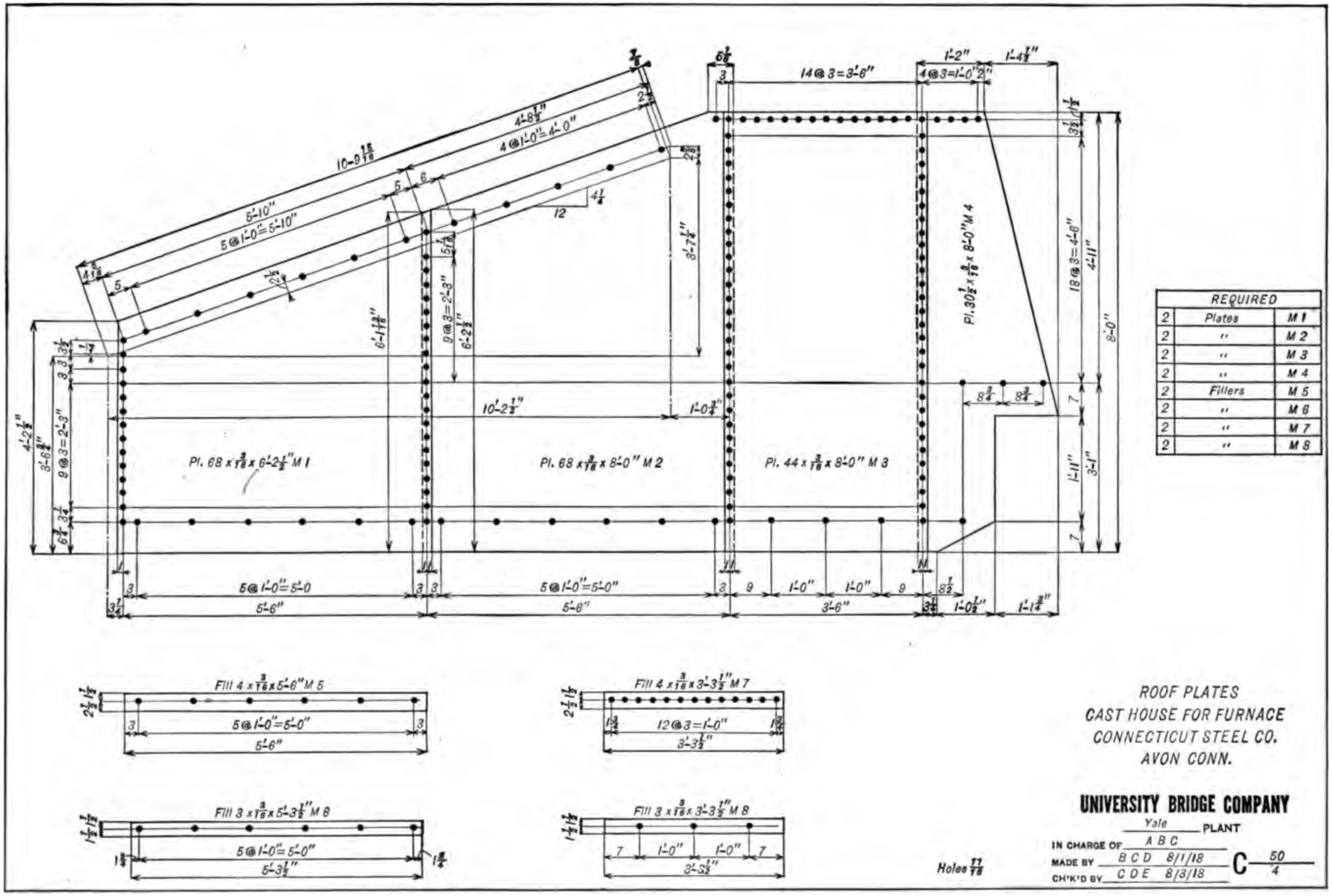


Fig. 148. Steel Roof Plates.

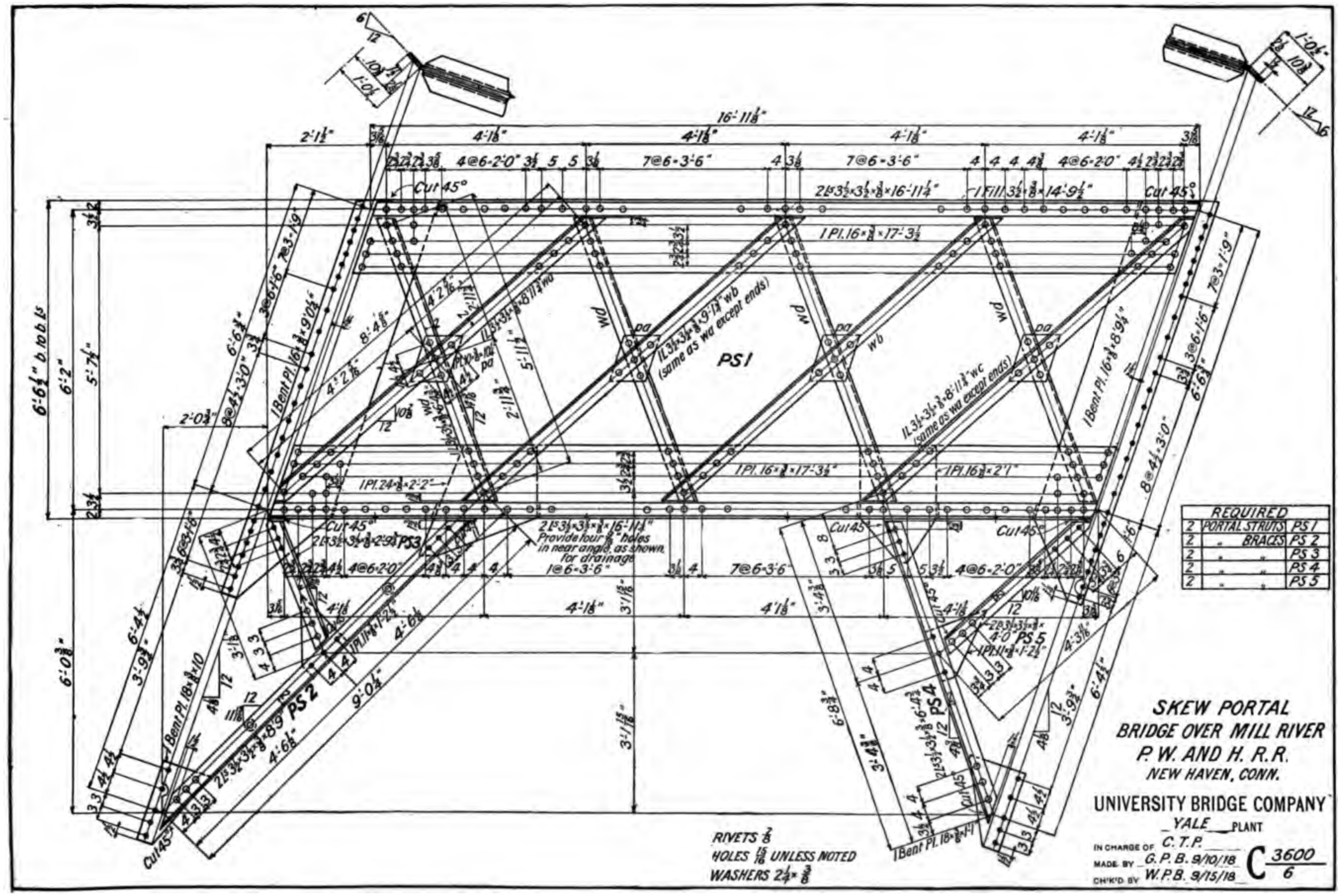


Fig. 149. Portal Bracing for a Skew Bridge.

the cover-plate faces of the end posts. There are only two angles to be determined which involve the use of angles in more than one plane; these are the angle of bend (B) and the skew angle (P) in the plane of the portal. Formulas for these angles are shown in Fig. 150 in terms of the truss angle (T) and the skew angle (H) in a horizontal plane.* The use of these formulas is illustrated by the determination of these angles in the portal of Fig. 149, using the following data:

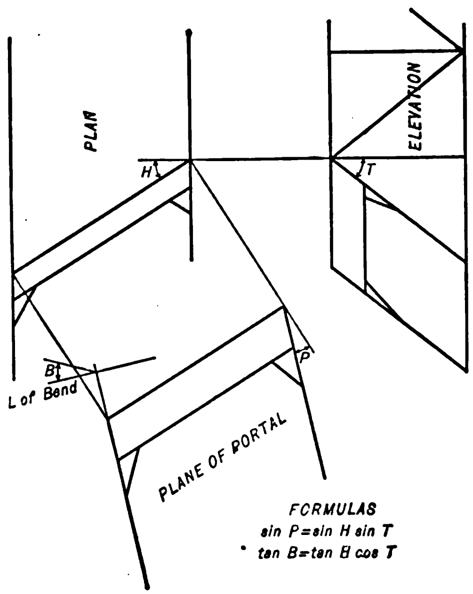


Fig. 150.

22′ 2½″ = the panel length of the truss.

29′ 0″ = the panel depth of the truss.

16′ 5″ = the distance c. to c. of trusses.

9′ 0″ = the lead of one truss in advance of the other, or the amount of skew, measured parallel to the axis of the bridge.

14′ 4″ = 16′ 5″ − 2(1′0½″) the distance c. to c. of working points measured at right angles to the axis of the bridge.

* For other angles see Conklin's "Structural Steel Drafting," John Wiley and Sons, Inc., New York; Turner's articles in Vol. 43 of the *Engineering News;* or Dannenberg's diagram in the *Engineering Record,* Feb. 17, 1912.

The functions of the truss angle (T) and the skew angle (H) are found as follows:

log 22′ 2½″	= 1.34652	log 9′ 0″	= 0.95424
log 29′ 0″	= 1.46240	log 14′ 4″	= 1.15635
log *tan T*	= 9.88412	log *tan H*	= 9.79789
log *sin T*	= 9.78390	log *sin H*	= 9.72572
log *cos T*	= 9.89978		

From $\sin P = \sin H \sin T$, log *sin H* = 9.72572
log *sin T* = 9.78390
log *sin P* = 9.50962

whence the skew in the plane of the portal has a slope of 4⅛ in 12.

From $\tan B = \tan H \cos T$, log *tan H* = 9.79789
log *cos T* = 9.89978
log *tan B* = 9.69767

whence the bend in the plate is 6 in 12.

The distances 6′ 0⅜″ and 6′ 2″ are determined by the allowed clearance; by multiplying them by *tan P* the distances 2′ 0¾″ and 2′ 1½″ are found. The working points of the upper and lower ends of the diagonals are located so that the horizontal panel lengths are equal. The two different lengths of diagonals from center to center of end rivets may be calculated in the usual manner after the proper horizontal distance between the end rivets of each diagonal has been found. The rivet spacing in the top chord from left to right should be made the same as the spacing in the bottom chord from right to left, in order to make the plates and angles interchangeable.

1. Eye bars, rods, and **miscellaneous material** are usually shown on special printed forms or else drawn on combination sheets, as explained on page 174 : 2.

CHAPTER XXV

ERECTION PLANS AND DIAGRAMS

SYNOPSIS: Outline or skeleton drawings are made to represent a portion or the whole of a structure to show the relation of the different members, and how these members are to be assembled at the site. The identification mark of each member is indicated upon the diagram for the use of the draftsman, the erectors, and others. The diagrams may be used also as progress record sheets.

1. **The plans and diagrams** of a proposed structure serve as keys to the structure. General dimensions, identification marks, and other information regarding the structure as a whole are thus presented to the draftsmen, the shopmen, the inspectors, the erectors, and the contractors for allied work. The diagrams are usually prepared as soon as possible after the design sheets are received in the drafting room. In some classes of work they cannot be completed until later, but the main outline is prepared and the missing dimensions and identification marks are added as the detailed drawings are made. Each member is usually represented by a single heavy line. These lines are not drawn far enough to intersect, but spaces are left to indicate the extent of each member. Often typical connections are shown on the diagrams. This insures greater uniformity when the drawings are made by several draftsmen and enables the engineers who approve the drawings to approve the types of connections before they have been incorporated extensively on the detailed drawings.

2. **The draftsmen** who make the working drawings obtain their information largely from the diagrams which are prepared in the drawing room, although they must supplement this information by data from the design sheets and from other available sources. As each working drawing is completed the draftsman should make sure that the identification marks are properly placed upon the diagrams. If the marks have been put on by another person he should verify them. These marks should also be checked by the person who checks the corresponding working drawing.

3. **The erector** uses the diagrams as a guide in assembling the different members in the field. For this purpose not only should every identification mark be shown, but sufficient notes and sketches should be added to insure the proper erection.

4. Diagrams must be prepared to give necessary information to **other contractors** who construct parts of the same structures or connecting structures. Thus a foundation plan must be drawn for the contractor who builds the foundations. A data sheet or crane clearance diagram must be prepared for the contractor who builds the traveling cranes for a building. This is usually based upon similar diagrams or upon charts furnished by crane manufacturers, but a new diagram is made for each structure and submitted to the crane manufacturer for approval to make sure that ample clearance is provided for the crane. Similarly, special data sheets may be required in order that proper provision may be made for machinery or shafting. Such data sheets may be made either by the structural draftsman or by the draftsman of the company which furnishes the machinery.

5. The diagrams of simple structures may be combined on a single sheet, but as the number of members shipped separately increases, the diagrams become more complex and many sheets may be required. A list of all the drawings on a contract is usually placed upon one of these

sheets. **Typical plans and diagrams** for different classes of work are illustrated below. A careful study of these diagrams should be made. They may serve as a guide when similar diagrams are being drawn for the first time.

1. **Plate-girder bridges** of single span may usually be completely represented by one diagram. Such a diagram for a through bridge is shown in Fig. 153. The plan of anchor bolts is shown at the bottom, a partial elevation with important dimensions is shown near the middle of the sheet, while the plan of the bridge at the top of the sheet shows the relative position of the girders, the floor system, and the lateral bracing. In this simple structure all the marks appear upon the plan which would therefore be sufficient for the erection of all the steel work. The more detailed elevation is added, however, in order to show the relative elevations clearly so that the foundations may be placed at the proper distance below the base of rail.

2. The erection diagram for the steelwork of a **truss bridge** is shown in Fig. 154. In bridge work the *far* truss is shown in the elevation, the members in the left half being detailed in the working drawings. These members are consequently marked "right" when "rights" and "lefts" are required (page 81:2). A combined cross-section and end view is drawn in sufficient detail to show the main dimensions and the relative elevations of the abutments, the floor beams, the stringers, and the rails. In the plan of the floor system, portions of the single-angle diagonals between the stringers are shown in a typical panel to indicate which way the angles are to be erected.

3. A simple **anchor-bolt plan** for a mill building is shown in Fig. 155. From this plan the contractor for the foundations can determine the number and the location of the piers which support the columns, the sizes of the corresponding column base plates and the column loads, the sizes and the spacing of the anchor bolts and the distances they are to be imbedded in the masonry. Dimensions to the extreme exterior of the building (in this case corrugated steel) are also given. The list of anchor bolts and washers which the contractor must set is summarized. This furnace building is left open all around for about 10 feet from the ground (see Fig. 156) and hence no doors are required. Small piers for door posts would be indicated upon the plan and the door posts shown in detail much as the columns are shown. Usually the small swedge bolts for door posts are placed after the steelwork is in position. A note to this effect should appear near the enlarged detail of the bolts. A note explaining who furnishes and sets the anchor bolts should be given unless uniform practice makes such a note unnecessary.

4. A typical **erection diagram for a mill building** is shown in Fig. 156; several typical detailed drawings for this same building have already been shown. For larger or more complicated buildings more than one sheet would be required, the plan of the roof and the elevations being placed on one sheet and the other plans on another sheet. This is particularly necessary when elevations of both sides and perhaps both ends are required. Note that portions of the girts are shown near a corner of the building to indicate whether the vertical legs are turned up or down; they are usually turned down except over windows, doors, or other openings. The column marks are repeated in the different views for the convenience of the draftsmen and others who wish to find the drawing of the column which shows what provision has been made for the connection of any specific member.

5. **A crane clearance diagram** for the same mill building is shown in Fig. 157. This shows the crane manufacturers just how much space is available for the crane and its appurtenances (see above).

6. Special erection diagrams are usually drawn for **corrugated steel.** These diagrams show the position of the sheets of different lengths and show how some of them are beveled in the field for the gable ends. Typical details are drawn to show the arrangement of the corrugated steel and the flashing at corners, windows, doors, cornices, ventilators, etc. These diagrams are usually drawn by experienced men, and it does not seem advisable to devote sufficient space in this book to treat the subject as fully as it should be treated, if at all.*

7. A portion of a typical **floor plan of an office building** is shown in Fig. 158. An experienced draftsman can usually make the plans of office buildings quite complete. He can determine the lengths of all main material and he can anticipate the details with sufficient accuracy to assign identification marks to all beams and girders, making sure

* For typical plans and details see either Ketchum's "Mill Buildings," or Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

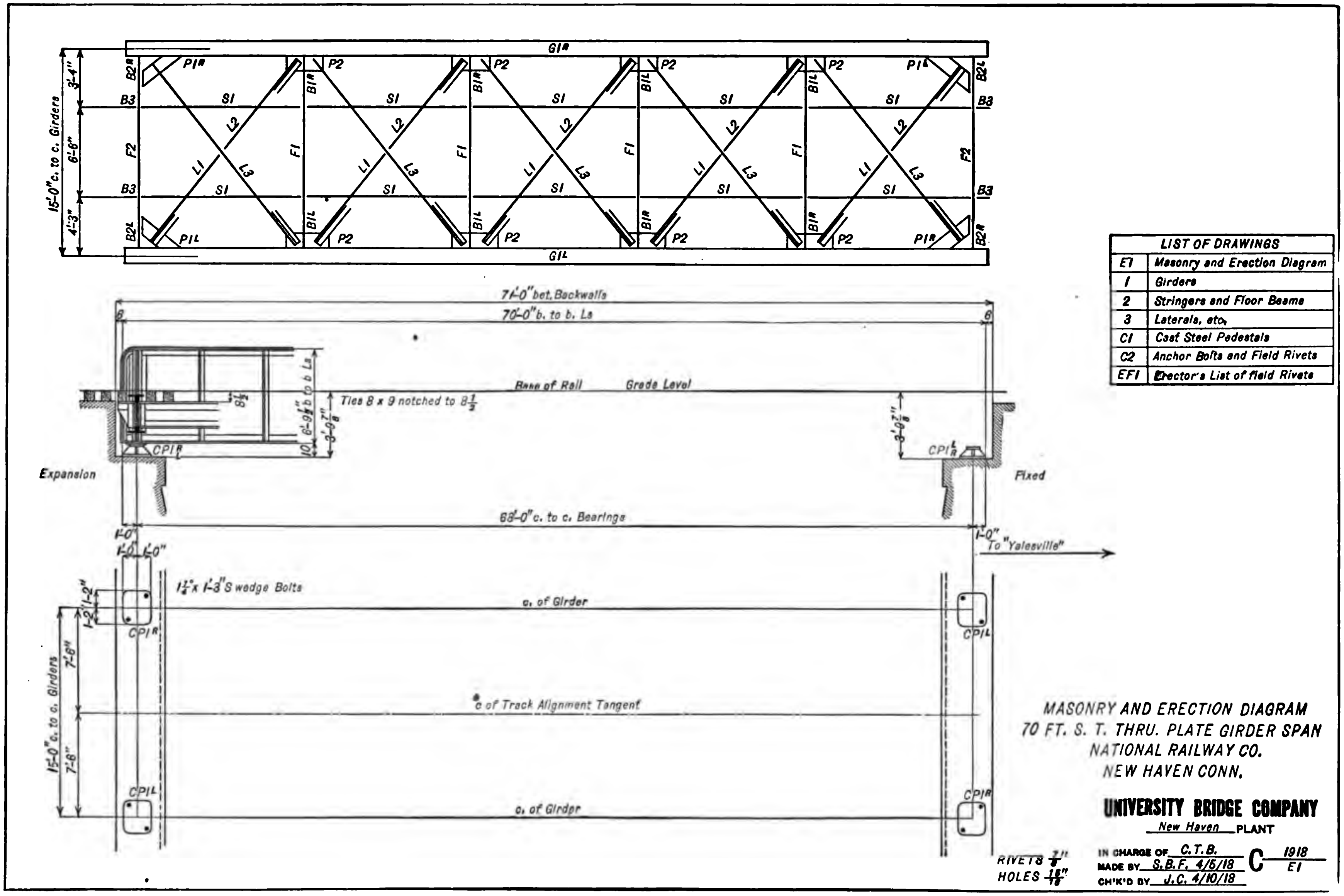


Fig. 153. Masonry and Erection Diagram for a Railroad Girder Bridge.

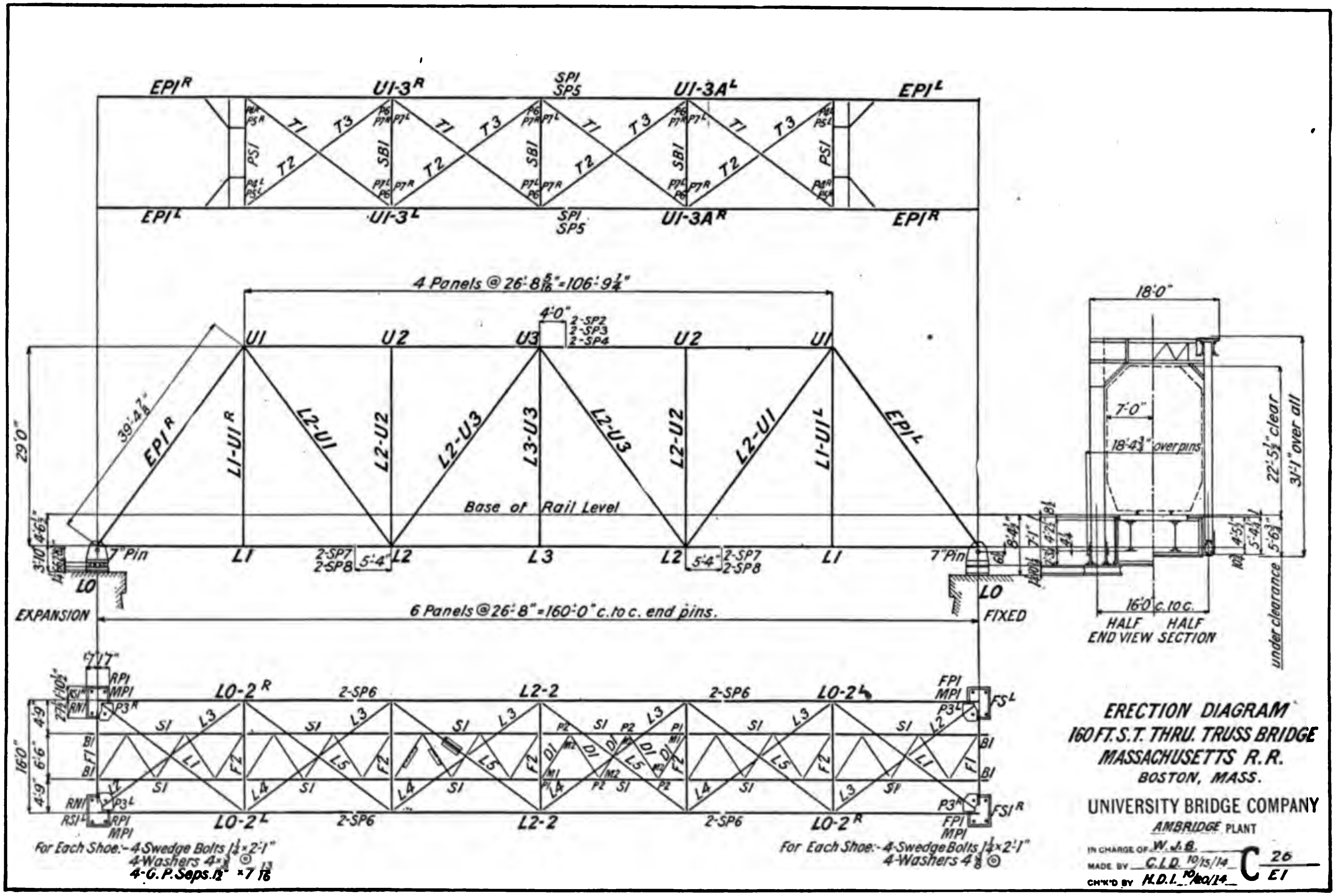


Fig. 154. Erection Diagram for a Railroad Truss Bridge.

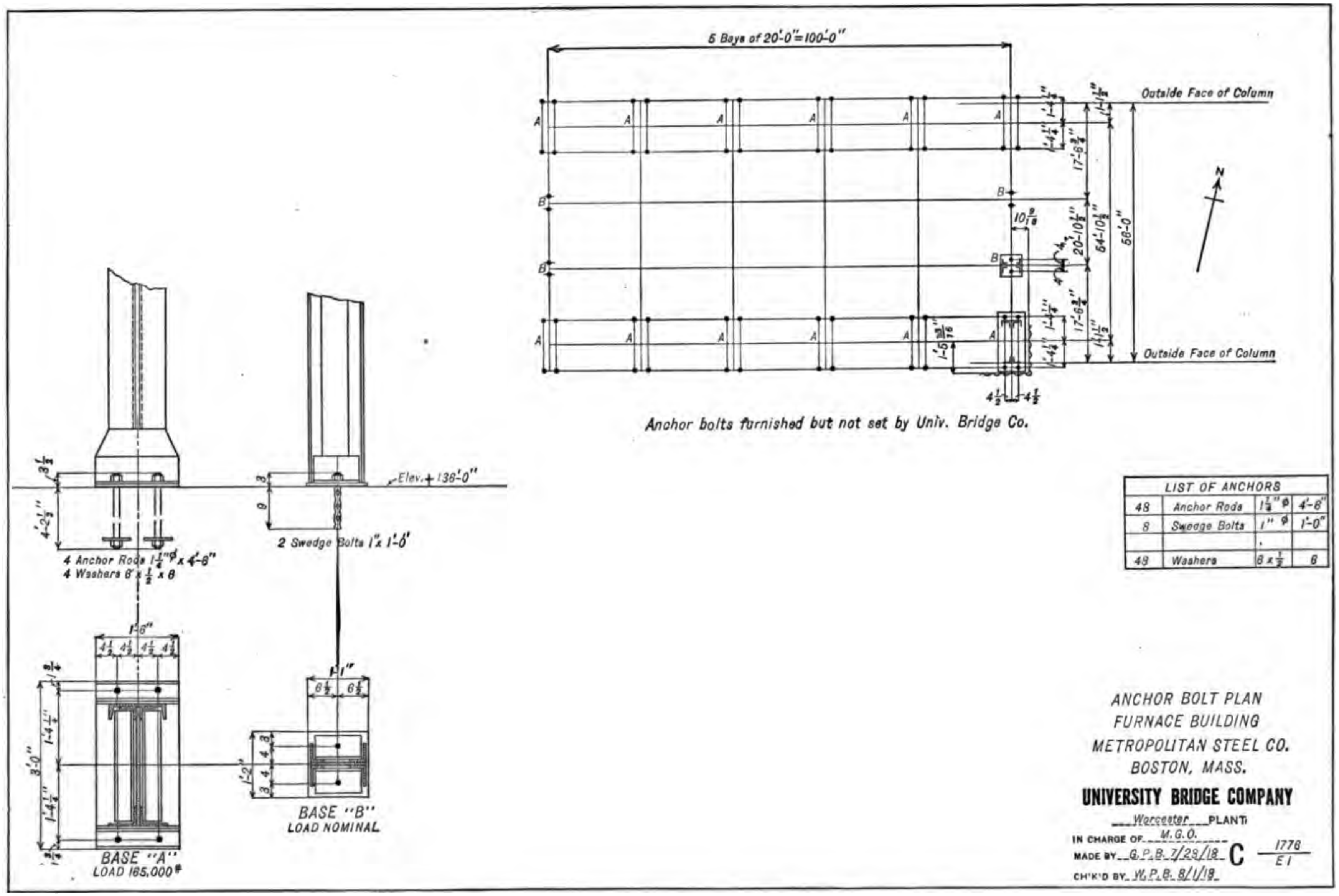


Fig. 155. Anchor-bolt Plan for a Mill Building.

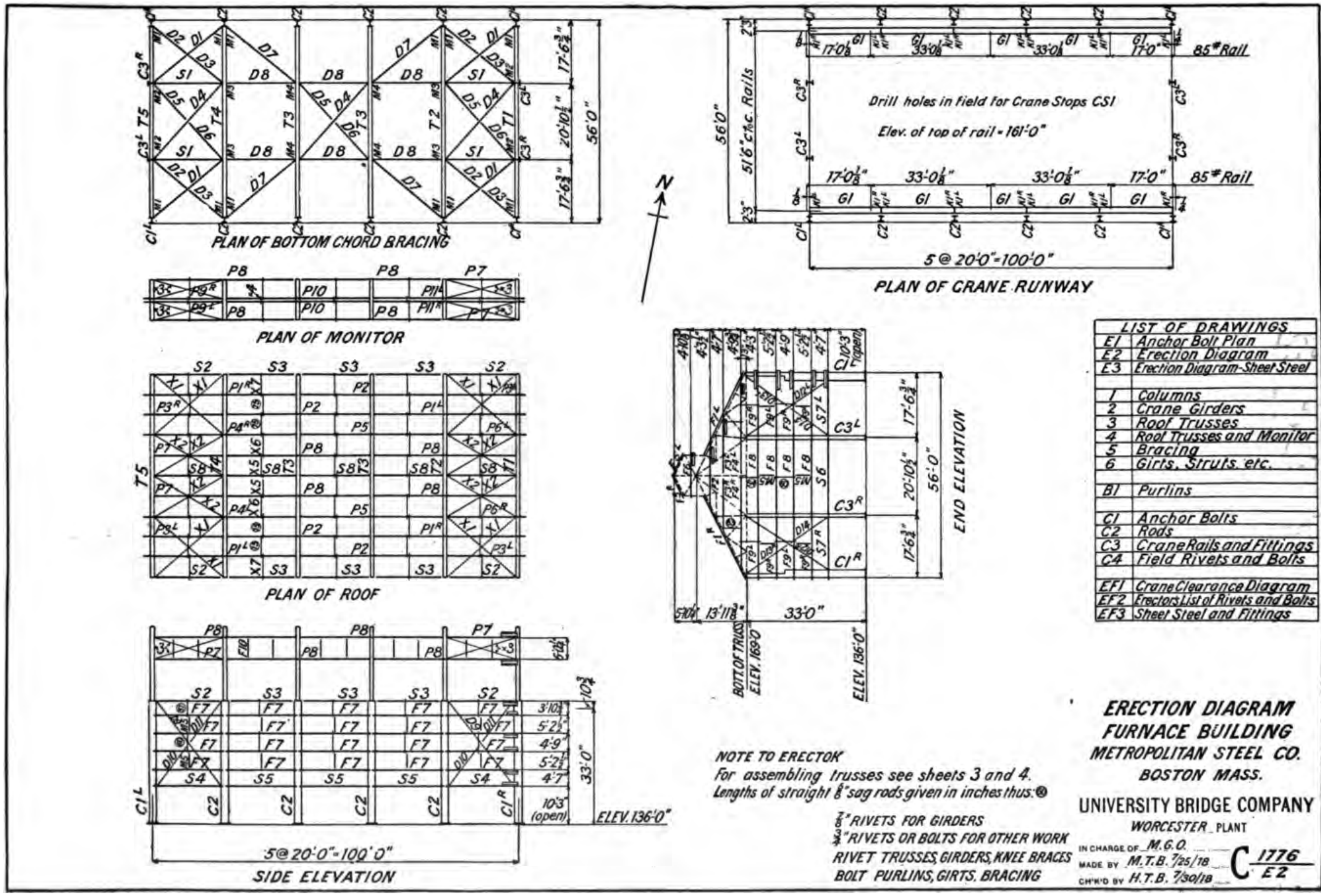


Fig. 156. Erection Diagram for a Mill Building.

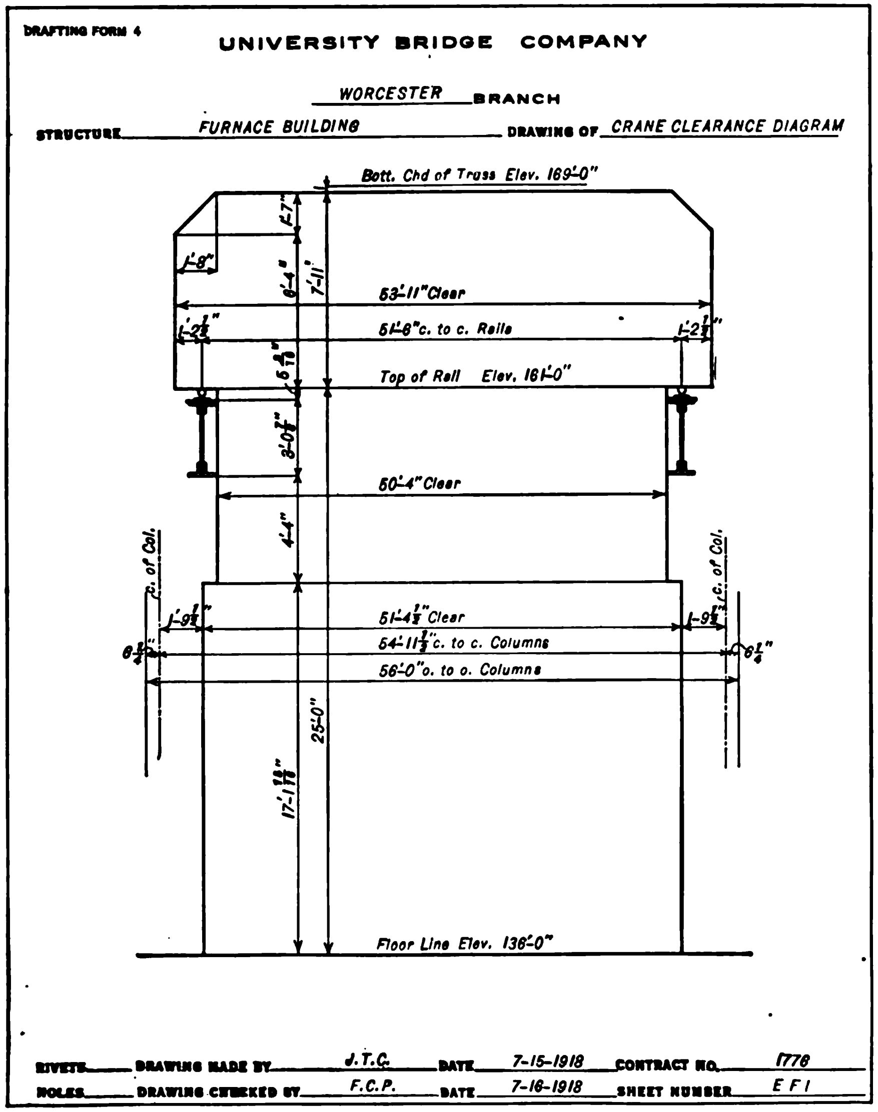


Fig. 157. Crane Clearance Diagram.

that beams which are alike are marked alike, and conversely that no beams are marked alike unless they are interchangeable. The columns are numbered consecutively, no two bearing the same number (see page 81 : 5). Typical connections may be shown on the diagrams. The advantages of having the plans carefully and completely made at the outset are many. First, the material order bills may be made with more assurance than if ordered from the design sheets without identification numbers. Secondly, the plans may be sent for approval at once and the main dimensions, typical connections, and special wall details may thus be approved before they have been incorporated in so many detailed drawings that any change would necessitate excessive alterations. Thirdly, if any necessary information is lacking the fact is usually discovered in time to obtain the complete data before it is needed. Fourthly, the making of the working drawings for the beams becomes routine work which can be done by comparatively inexperienced draftsmen with very little supervision.

1. **Method of Procedure.** — An office-building plan is drawn in about the following order. First the column centers are located and then the column sections are shown conventionally. Although considerable work is involved in showing each column it seems worth while to indicate clearly the type of each column and the way it is turned. Next the beams may be drawn and the corresponding dimensions recorded. All openings for stairs, elevators, stacks, etc., are indicated conventionally by light diagonal lines so that no floor will be constructed in these spaces. Tie rods are indicated wherever they are required to resist the thrust of the floor arches as indicated upon the design sheets (see page 20 : 3). Each beam section may now be billed on the diagram. Much repetition may be avoided through the use of the abbreviation "do" for "ditto"; ditto marks (") should never be used because they are not sufficiently distinctive. Care must be taken not to insert any other beam section between one beam section and a corresponding "do." The diagram should show clearly the way the flanges of channels turn. This may be done by means of small sectional views, or by indicating a portion of the channel flange. A more convenient method is to turn the channel mark (⊔) to indicate which way the flanges face, as illustrated near the lower left-hand corner of Fig. 158 where the channels numbered

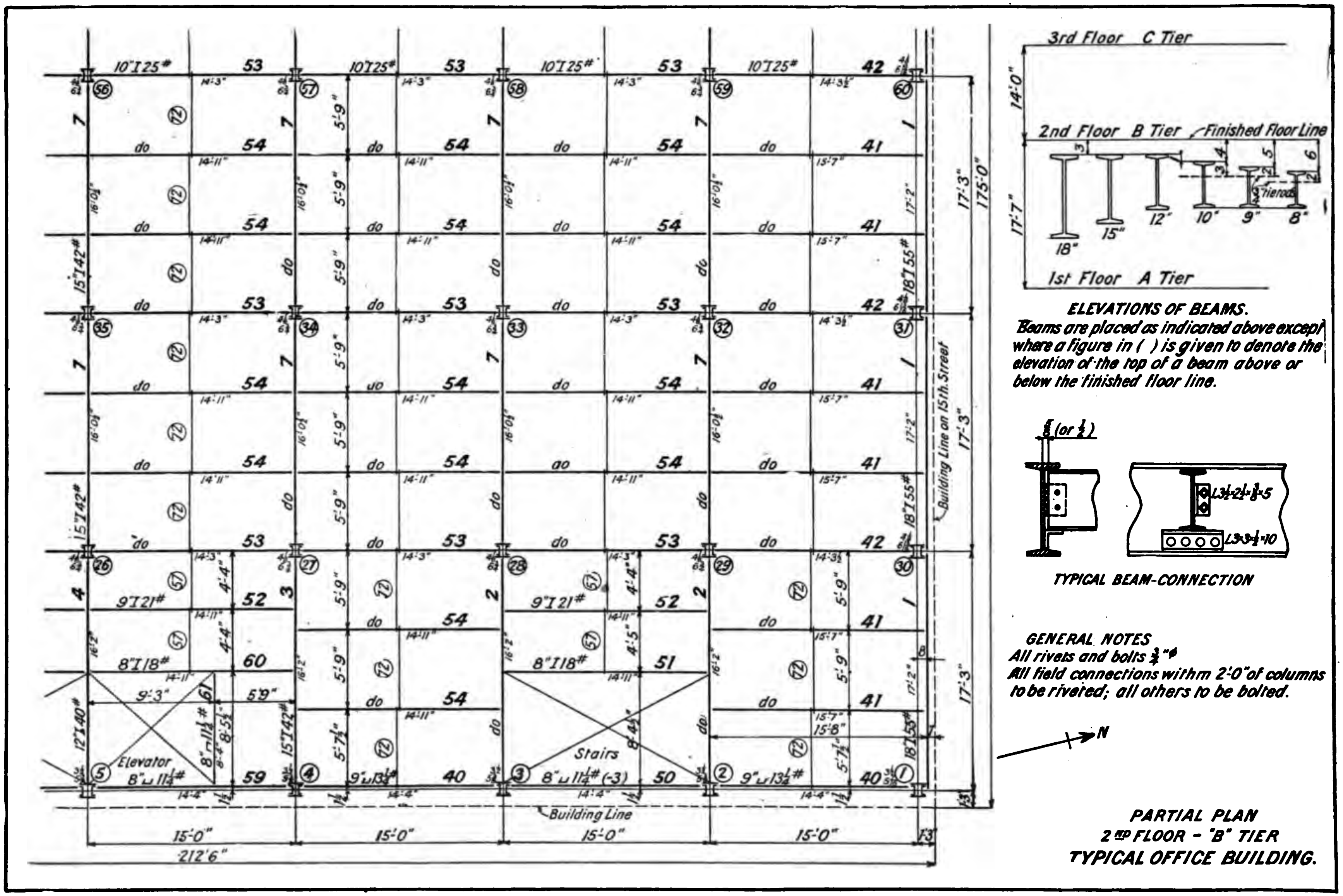


Fig. 158. Partial Floor Plan for an Office Building.

59 and 61 are billed. The relative elevations of all beams and tie rods should be indicated. A general sketch may be drawn for most beams, while the elevations of the tops of exceptional beams above or below the finished floor line may be indicated by dimensions in parentheses. The type of connections for beams to beams and beams to columns should be determined and preferably shown on the plan. Since the lengths of the beams which connect to the columns depend upon the sizes of the columns it is convenient to place near each column small figures to indicate the distances from the center of the column to the outer faces. Thus for the channel columns shown, one figure gives the distance from the center to the outer face of the channel (constant for a given depth of channel), and also the distance from the center to the outer face of the cover plates (varies with the thickness of the plate and also with the depth of channel). After these figures are completed and the type of connection determined, the length to be ordered may be placed on each beam. The experienced draftsman can now quite easily assign to the beams the numbers of the identification marks. The floor number or letter of the complete identification mark (see page 81 : 5), is given once for all in the title and need not be repeated on each beam. The lengths of tie rods should be indicated on the plan. Some companies assign a letter with consecutive numbers to the rods, as X 1, X 2, etc. A better method is illustrated in the plan shown, where the identification mark in the circle is the length of the rod in inches. See page 82 : 3. It is unnecessary to mark every rod on the plan when a single mark between continuous lines of beams will suffice.

1. Plans for **different floors** may be combined when quite similar. The column dimensions and beam lengths may differ but these differences may be indicated without drawing separate plans. Ordinarily, separate plans are drawn for the first and second floors and the roof, the intermediate floors being combined in a single plan.

2. **A column schedule** is often prepared for use in the drafting room, although it is not indispensable to the erector. One form of column schedule is shown in Fig. 160. One space is laid off horizontally for each column, and the story heights are plotted and dimensioned vertically. Heavy lines are drawn to indicate the milled ends of the column sections at the splices and at the extreme bottoms. The line at the bottom is shown broken, when the column bases are not all at the same elevation. The component material for each section is given, and also any fillers required at the splices. The cap plates are indicated clearly by shaded spaces so that the proper main material will be ordered short to allow for the thickness of the cap plates. The reinforcing plates may be given just below the cap plates. The loads or the column areas are sometimes given, but this is usually unnecessary. If one whole column from basement to roof is like another in section and in length, it may be referred to the other in order to avoid needless duplication. A typical column splice is often shown on this sheet so that it may be approved before the detailed drawings are far advanced. The column schedule may be made to serve as an index to the drawings upon which the different columns are detailed; the drawing number may be placed at the top of each rectangle as shown.

3. A tabular **record of drawings** is usually made for each contract. This provides for the initials of the draftsman and the checker for each drawing with dates showing when the drawing is completed and checked. Dates are recorded to show when prints are sent for approval and when approved, and when prints are issued to the different shops, to the inspector, to the erector, and to others interested. These tabular records are arranged to give the necessary information regarding sheets, but they do not give complete information regarding the members. In office-building work especially there are so many similar drawings, such as columns and beams, that it is difficult to make sure that the drawings are prepared in a logical order unless supplementary records are kept. On account of limited storage facilities near the site of an ordinary office building all material must be shipped as required. The squad foreman should guard against having the roof beams and columns detailed while some on the first tier remain untouched. Blueprints of the plans and the column schedule may be used to excellent advantage as progress record sheets, as explained in the following paragraphs.

4. **Progress Record for Beam Drawings.** — A blueprint of one of the floor plans is assigned to a draftsman who is to prepare the working drawings of the beams. As the drawing for each beam is completed, the draftsman should mark the plan with a distinctive color, say yellow. An ordinary check mark is not sufficient to clearly show his progress.

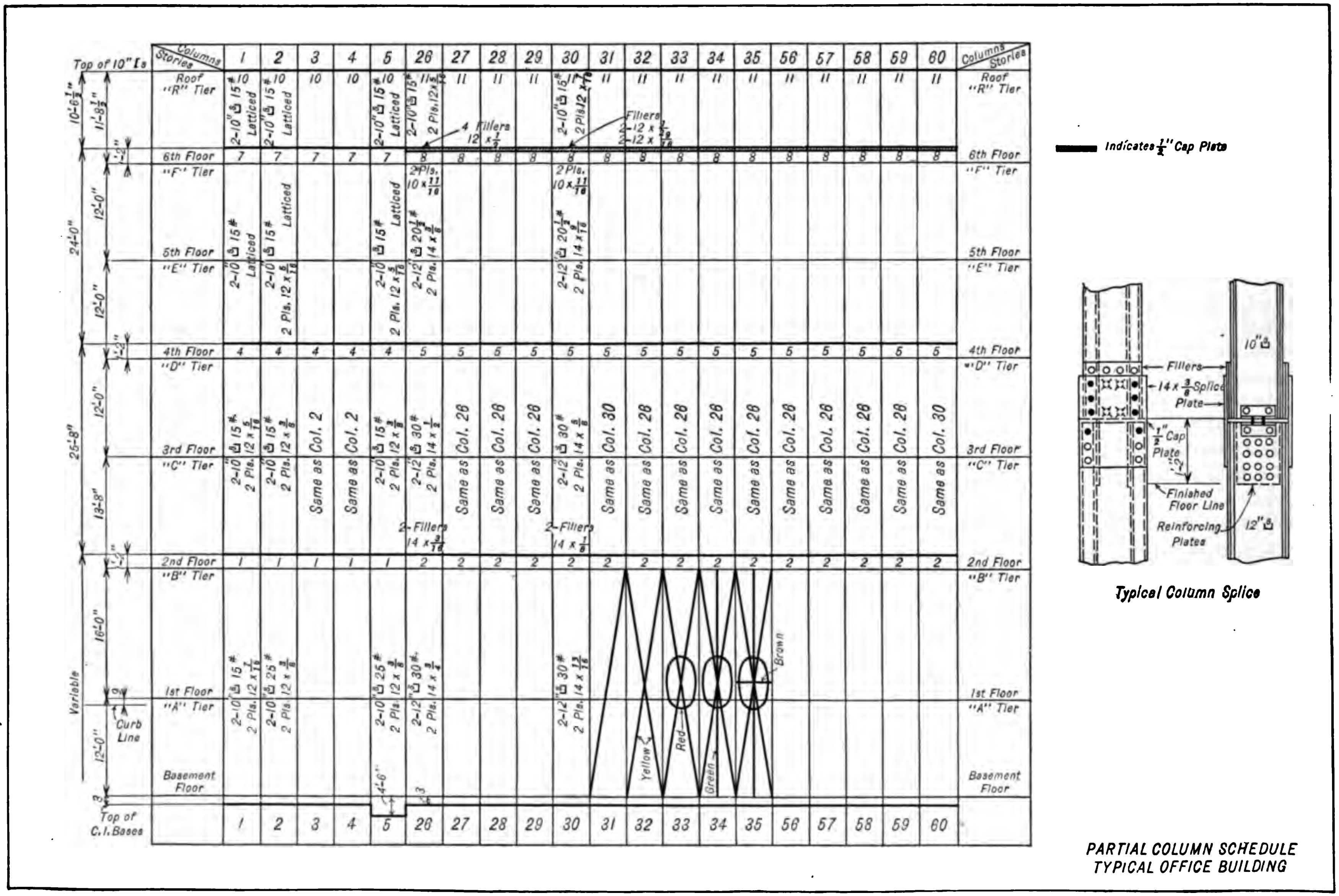


Fig. 160. Partial Column Schedule for an Office Building.

He should draw the crayon the full length of the beam so as to completely obscure the white line on the blueprint. In this way the whole tone of the print is changed from white to yellow, and the squad foreman or the chief draftsman can tell at a glance without interrupting the draftsman what proportion of the beams are detailed. Furthermore, when the draftsman has completed the drawings he can scan the print and easily detect any beam which he may have overlooked because a lone heavy white line will stand out conspicuously among the yellow lines and the fine white dimension lines. It is desirable to have all beams on one plan detailed by one man in order to insure uniformity of details and to avoid duplication. If the available time will not permit this, the work may be assigned to more than one draftsman, but the division should be made definite so that no beam in one portion is like any beam in the other portion. This may often be accomplished by assigning the wall beams to one man and the intermediate beams to the other. If a plan represents one or more floors, the beams for all the floors are usually drawn at the same time. Many of the beams may be combined upon the same sketch provided different marks are assigned the beams of different floors as in *C* 17, *D* 17, etc., Fig. 92. Another blueprint of each plan is given to the checker who marks the beams as he checks the corresponding drawings. He uses a different color of crayon, say red, but he marks the plan in the same manner as the draftsman in order to obscure the white lines of the beams.

1. A print of the column schedule may be used for a **progress record for the columns.** All records for one contract should be made upon the same print. This may be posted upon the wall if accessible space is available or it may be filed near the squad foreman's desk. It is convenient to have this record near the tabular record of drawings, for the two are frequently used together. Almost any amount of information can be recorded upon the print of the column schedule to satisfy the requirements of the men in charge of different drafting rooms. Significant colors and arrangements may be used for different needs. Yellow and red are most distinctive while green, brown, or black may be used when additional colors are desired. The following suggestions may be used as a guide. Each column section is represented by a rectangle bounded by adjacent vertical lines and heavy horizontal lines (see above). Any system of marks should be confined to one of these rectangles, the other rectangles being marked similarly. The draftsmen should use the same color of crayon (yellow) as in beam work. In order to indicate that he is working upon a certain column or group of columns a draftsman should draw a diagonal line across the rectangle of each column of the group so that no one else will duplicate his work (see column *AB* 31, Fig. 160). When he has completed the drawing and it is ready to be checked he draws the other diagonal (*AB* 32). The checker adds a red circle or oval to each column as soon as it is completely checked (*AB* 33). A green vertical line may be drawn through the center of the rectangle when prints have been sent for approval (*AB* 34) and a horizontal brown or black line may be drawn across the center of the rectangle when prints have been issued to the shop (*AB* 35). Other records may be made by means of horizontal lines drawn across the rectangle above or below the center, using any of the above colors provided all horizontal lines with different significance are of different colors. By drawing the lines entirely across each rectangle continuous lines are formed when all the columns are marked. This simplifies the detection of any omissions because a break in a continuous line is conspicuous.

CHAPTER XXVI

MATERIAL ORDER BILLS

SYNOPSIS: Preliminary lists of material are usually prepared in the drafting room for each contract before the drawings are made. These order bills are sent to the Order Department where the material is ordered from the rolling mills.

1. **Purpose.** — The chief function of a structural steel company is to build structures from steel which is already rolled into common commercial shapes. The drawings show how such shapes are cut, punched, and combined to make members which are subsequently connected to form complete structures. Some companies have their own rolling mills but most companies procure their steel shapes elsewhere. In either case the material must be ordered from the rolling mills. Usually it takes so long to have an order filled that it is impractical to wait until the working drawings are made before placing the order; often both the drawings and the templets can be made during the interval that elapses between the placing of the mill order and the delivery of the material. It is therefore imperative that all important material be ordered as soon as possible after a contract is made.

2. The original **lists of material** are prepared in the drafting room by men who are familiar not only with drafting room methods but also with the requirements of the Order Department. On these lists the material is classified according to types of members, as for example, columns, trusses, beams, etc. In the Order Department new lists are prepared to meet the requirements of the rolling mills. On these lists the material is summarized and reclassified according to sections and lengths and an item member is assigned to each different item. Short plates and angles are usually ordered in multiple lengths to be cut to the desired lengths after they are received. Most companies carry a certain amount of material in stock for immediate use. The stock yard is under the jurisdiction of the Order Department, where the necessary material is ordered to keep the yard supplied with the desired amount of stock. All additions and subtractions should be so recorded that one can tell at any time just what material is in stock. In the Order Department it is determined whether an item shall be taken from stock or ordered from the mills.

3. **Methods.** — When market conditions are such that there is likely to be considerable delay in filling an order it is especially urgent that all main material be ordered at once. Capable men are assigned to this work in order that it may be done as efficiently and expeditiously as possible. These men can often foresee the details of construction with sufficient accuracy to enable them to order much of the material by referring to the design sheets. In more complex work they may have to lay out some of the details which determine the lengths of main material. The lengths of angles of light trusses may often be determined with sufficient accuracy for ordering by scaling a drawing without stopping to calculate the lengths. For this purpose a draftsman begins a working drawing; he lays down the working lines to scale and draws the outlines of the angles. The angles are usually ordered about $1\frac{1}{2}''$ longer than the scaled lengths, to provide for inaccuracies. After the lengths are scaled the drawing may be laid aside until later, if desired. The beams and columns of office buildings can be ordered more satisfactorily after the erection plans and the column schedules have been prepared, as explained in the preceding chapter.

1. **Miscellaneous material** other than structural steel shapes should also be ordered as early as possible so that all will be available when needed. Some of the more common examples of such materials are: eye bars, rails and rail fittings, pins, forgings, castings, pipe, corrugated steel and other roof and side coverings, lumber, windows and doors, and hardware.

2. The material order bills are divided into **two parts.** The first part is the main list of material and the corresponding description which are made in the drafting room in accordance with this chapter. The original sheets are written with copying pencils; these originals are sent to the Order Department. A carbon copy is also made for use in the drafting room. After the corresponding mill orders are written in the Order Department, the second or central part of the original bills under "Mill Order" (Fig. 163), is filled in by the Order Department; copies of the sheets are then made in a copying press, and the originals are returned to the drafting room. The central portion shows the item numbers and other information required by the men who itemize the shop bills (page 169 : 1). Nothing but the item number is given unless the number of pieces, the section, or the length differ from those shown in the main part of the bill; all the differences are recorded. The section seldom differs except for material to be planed or recut at the shop. The length may differ by a small amount recorded in the column headed "+ ins.," or by a large amount when material is in multiple lengths. The number of pieces may differ when pieces are ordered in multiples (item 31, Fig. 163), or when similar items are combined (item 1). In the latter case, the number of pieces and the length are recorded opposite the first item, the others being referred to this by the words "see above," or "see page ——"; the item number is repeated.

3. Each draftsman must make his **drawings conform to the material ordered,** and he must report any instance where this is not feasible. Slight variations are of less consequence when the material is taken from stock or cut from the multiple lengths than when the material is cut to the desired lengths at the mill. The draftsman should, therefore, consult the original bills after they are returned; the carbon copies are used only while the originals are held by the Order Department.

UNIVERSITY BRIDGE COMPANY

________ BRANCH

STRUCTURE TYPICAL OFFICE BUILDING – 1ST SHIPMENT MATERIAL ORDER BILL

Material: No. of pieces	Material: Section	Material: Length ft.	Material: Length ins.	+ ins.	Mill Order: No. of pieces	Mill Order: Section	Mill Order: Length ft.	Mill Order: Length ins.	Item	Description
			1st Tier Columns – A B							
2	10" ⊏s 15#	29	5	3/4			29	5 3/4	4	Col. 1
2	Pls 12 7/16	29	5	3/4			29	5 3/4	32	
6	10" ⊏s 25#	29	5	3/4			29	5 3/4	3	Cols. 2, 3, 4
6	Pls 12 3/8	29	5	3/4			29	5 3/4	34	
2	10" ⊏s 25#	33	8	3/4			33	8 3/4	2	Col. 5
2	Pls 12 3/8	33	8	3/4			33	8 3/4	33	
24	12" ⊏s 30#	29	5	3/4	30		29	5 3/4	1	Cols 26-29, 32-35, 56-59
24	Pls 14 3/4	29	5	3/4			29	5 3/4	30	
6	12" ⊏s 30#	29	5	3/4		see above			1	Cols. 30, 31, 60
6	Pls 14 13/16	29	5	3/4			29	5 3/4	29	
10	" 14 3/8	1	6 3/4		2		39	0	31	Splices 10" Cols.
30	" 14 3/8	2	0 3/4							" 12" "
			2nd Floor Beams – B Tier							
3	18" Is 55#	17	2						10	Beam 1
3	15" Is 42#	16	2						11	" 2, 3
8	15" I 42#	16	0 1/2						12	" 7
1	12" I 40#	16	2						13	" 4
6	10" I 25#	15	7						14	" 41
14	do	14	11						15	" 54
3	do	14	3 1/2						16	" 42
9	do	14	3						17	" 53
2	9" Is 21#	14	11						18	" 52
2	8" Is 18#	14	11						19	" 51, 60
2	9" ⊏s 13 1/4#	14	4						20	" 40
2	8" ⊏s 11 1/4#	14	4						21	" 50, 59
1	8" ⊏s 11 1/4#	8	4						22	" 61
	3/4" ⌀ 200 lin. ft.				10		20	0	53	Tie Rods

BILL MADE BY G. P. B. DATE 2/17/1919 CONTRACT NO. ____

BILL CHECKED BY W. P. B. DATE 2/21/1919 PAGE NUMBER ____

Fig. 163. Material Order Bill.

4. **The arrangement** and the description of the pieces billed should be such that the material for any members may be easily found and identi-

fied. The material for similar members should be listed together under a separate heading, as the columns or the beams of Fig. 163. A more detailed description should be given opposite each item of the bill whenever this will simplify identification. In some cases the shipping mark, if known, can be used to advantage; in other cases it may be better to note the position of the pieces as "Flange," "Stiffeners," "Base," "Crane Seat," etc. In each group the material for the main section should be listed first, followed by the less important pieces. These should be listed in a systematic order to facilitate locating them later. As a rule the detailed material for the end connections should be listed before that for the intermediate connections; if the position of a member on the drawing can be anticipated, the material at the bottom end or the left hand end should be billed first.

1. **Shipments.** — When contracts are divided into different sections and members are to be shipped accordingly, the order bills should be marked "First Shipment," or similarly, and all material should be grouped under the proper shipment. The last page of bills for each shipment should be marked "Complete." Each tier of office-building columns and the corresponding beams and girders should be kept separate from the others.

2. The pages of material order bills should be **numbered** consecutively from 1 up. It may be preferable to number the pages for different shipments in different series, as 101, 102, etc., for first shipment and 201, 202, etc., for the second.

3. On the first page or on a separate sheet should be given information regarding the **specifications,** the grade of steel, the nature of any tests which are to be made, the kind of oil or paint to be applied at the mill, and the name of the inspector.

4. All material which is to be **shipped direct** to the site should be clearly noted. Such material as corrugated steel, lumber, etc., need not, as a rule, be sent to the structural plant.

5. All **changes** in material orders should be sent to the order department on "change slips," or "change orders." These should cover all changes or cancellations in the original orders whether due to errors or to changes in design. Such changes should be reported at once so that if possible they may be made at the mill before the material is cut. Change orders should be made out for any material which is left over after the itemizing is complete (page 169 : 1). In this way extra material may be released, and made available for other contracts.

6. **Ordered lengths.** — The main part of the order bill should show the material as it will appear on the drawings. Any increase in length which is required on account of milling, recutting, or bending at the shop, should be indicated in the column headed "+ ins." The lengths which are actually ordered from the mill should be in multiples of ½ inch wherever possible. Many of the shorter pieces, particularly plates and angles, are ordered in multiple lengths. The steel shapes are cut at the mill as they come from the rolls still hot. It is not practicable to measure the lengths with great precision and a "mill variation" in length is allowed. Hence it is inconsistent to order material in sixteenths or eighths. Some of the more important points to be considered in ordering material will now be taken up for the different shapes.

7. **Beams.** — Because of the difficulty and the waste incident to cutting, I-beams and channels are usually ordered so that they can be used as they are received from the mills, unless they are to be milled (page 31 : 1). The standard mill variation in length for overrun or underrun is ⅜ inch. Greater allowance is made if feasible so that a beam need not be recut in the shop even though it overruns more than ⅜ inch (see page 88 : 1). Beams which frame between other beams are usually ordered about 1½ inches less than the distance between the *centers* of the webs of the supporting I-beams or channels. Beams which frame between girders or columns are usually ordered about 1 inch less than the *clear distance* between the supports. Runway beams which support the wheels directly should be milled to give about ⅛-inch clearance between the ends of adjacent beams. I-beams and channels which form parts of columns or chord members are often milled on the ends after they are assembled in order to give uniform bearing. Beams to be milled should be ordered long enough to permit recutting. If milled at one end ½ inch is added; if milled at both ends ¾ inch is added. This increase is indicated in the column headed "+ ins.," and the order department takes this into account in preparing the mill order. Beams used as purlins are usually ordered about 1 inch less than the distance center to center of trusses, the length being taken to the nearest ½ inch. This applies also

to angle, tee, or Z-bar purlins. I-beams and channels which are to be sawed diagonally at the shop should be ordered about 1 inch long. Short I-beams so cut may be ordered in multiple lengths to save waste (compare Fig. 44), allowing about $\frac{1}{4}$ inch waste for each saw cut.

1. **Angles** can be recut at the shop more easily than beams, hence it is not so important to use the exact lengths that come from the mill. The angles are not likely to underrun the ordered lengths but they may overrun as much as $\frac{3}{4}$ inch. Better results are obtained when the angles are sheared to the proper lengths at the shop. The main flange or chord angles of girders, columns, trusses, etc., are usually ordered about $\frac{3}{4}$ inch long to make recutting necessary whether or not these angles are to be milled. Angles which are to be milled at one end only are ordered $\frac{1}{2}$ inch long. These increases are indicated in the column headed "ins." Stiffening angles need not be ordered long unless they exceed $\frac{5}{8}$ inch in thickness when they must be milled, or unless they are crimped. Two or more angles of the same length may be cut from one piece, provided the sum does not exceed one car-load length of 30 or 35 feet. All angles less than 3 feet long are ordered in multiple lengths; angles more than 3 feet long are seldom combined unless they are of the same length or so nearly the same that the length of the longest may be used for each. When more than two pieces are ordered in multiple, the length should be increased to allow for waste; $\frac{1}{4}$ inch should be added for each cut, and usually enough more to bring the total to an even inch. Crimped angles must be increased in length by the depth of each crimp (page 97:1) and also by $\frac{1}{2}$ inch for each crimp to provide for recutting after crimping. Angle purlins are ordered about 1 inch less than the distance center to center of trusses, the length being taken to the nearest $\frac{1}{2}$ inch. Bent angles should be ordered about 3 inches long; curved angles should be ordered about 6 inches long.

2. **Plates.** — In this book no distinction in terminology has been made between plates of different sizes and forms. In ordering material from the mill, however, more specific terms must be used, according to the handbook of the steel company from which the material is ordered. "Flats" and "Universal Mill" or "edged" plates are rolled between vertical rolls as well as between horizontal rolls; these plates have rolled edges as distinguished from "sheared plates" which are rolled between horizontal rolls only, and then sheared to the desired widths. Flats include all widths up to 6 or 7 inches, Universal Mill (U. M.) plates from 7 or 8 inches up to 36 or 48 inches, and sheared plates from 24 inches up to 126 or 132 inches. Plates may be sheared less than 24 inches if desired. Up to 48 inches either U. M. or sheared plates will be furnished at the option of the manufacturers unless one or the other is specified. Flats are rolled in widths which are multiples of $\frac{1}{2}$ inch with a mill variation of $\frac{1}{8}$ inch. Widths less than 3 inches are rolled in multiples of $\frac{1}{4}$ inch. U. M. plates are regularly rolled only in widths which are multiples of 1 inch, with a mill variation of $\frac{1}{8}$ inch. Sheared plates may be ordered in widths which are multiples of $\frac{1}{4}$ inch with a mill variation of $\frac{1}{4}$ inch; preferably these plates should be ordered in multiples of 1 inch. Some widths are used more commonly than others, and except on large orders it is better to use these common sizes as far as practicable. Each company should provide its designers and draftsmen with a list of preferred sizes including the sizes carried in stock. The extreme lengths of plates vary with the cross sections.* The lengths of U. M. plates are not likely to underrun the ordered lengths but they may overrun $\frac{3}{4}$ inch. Sheared plates may underrun $\frac{1}{4}$ inch or overrun $\frac{1}{2}$ inch. Flats over 20 feet long may underrun $1\frac{1}{4}$ inches or overrun $\frac{5}{8}$ inch; shorter flats will vary only about one half these values. Plates are usually recut at the shop except in special cases; for example, the webs and the cover plates of girders are often used as delivered. Such webs should be ordered from $\frac{1}{2}$ to $\frac{3}{4}$ inch less than the extreme lengths back to back of end angles, or if spliced, $\frac{1}{2}$ to $\frac{3}{4}$ inch less than the distances between the centers of splices. No individual portion of a web should weigh more than 3000 pounds. Full-length cover plates are usually ordered about $\frac{3}{4}$ inch long for recutting; others need not be recut provided the rivets are spaced to allow for variation (page 106 : 2). Cover plates are usually specified U. M., particularly if exposed to the elements, for the rolled edges do not corrode so quickly. Some of the heavier gusset plates are ordered as "sketch plates," i.e., they are cut to irregular shapes at the mill according to dimensioned sketches. There is an extra charge for this work which will usually offset any saving in material

* See the handbooks of the different steel manufacturers, or Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Company, Inc., New York.

unless the plates are over 3 feet long and $\frac{5}{8}$ inch thick. Sketch plates should only be ordered upon the approval of an experienced man. Gusset plates may often be cut from multiple lengths with little or no waste, provided the diagonal cuts extend entirely across the plate, as in Fig. 44. Advantage should be taken of this fact in ordering the material. Multiple lengths should not be ordered too closely; it is better to allow $\frac{1}{4}$ inch extra for each cut and to increase the total length to an even inch. Plates which are to be planed should have due allowance made for waste. The widths should be increased $\frac{1}{4}$ inch for each edge planed; the thicknesses should be increased $\frac{1}{16}$ inch for each side planed; for plates over 3 feet square the latter should be increased according to plant standards.

1. **Lattice bars,** short **tie or sag rods, gas pipe** and some other miscellaneous pieces are usually ordered according to the approximate total number of linear feet (Fig. 163); these lengths can be converted to multiple lengths by the Order Department.

2. **Rails** for crane runways, etc., should be ordered according to standard lengths. Odd lengths may be ordered for one end of a runway to provide the proper total length. Rails which weigh over 60 pounds per *yard* are usually furnished in 33 foot lengths; lighter sections come in 30 foot lengths. Specify "standard punching in ends" if such spacing (page 317) is required; this punching is done at the mill. All special punching must be done at the shop. If there is no special punching or cutting the rails can usually be shipped directly from the mill to the site; if this is desired it should be so noted on the order bills.

CHAPTER XXVII

SHOP BILLS AND SHIPPING BILLS

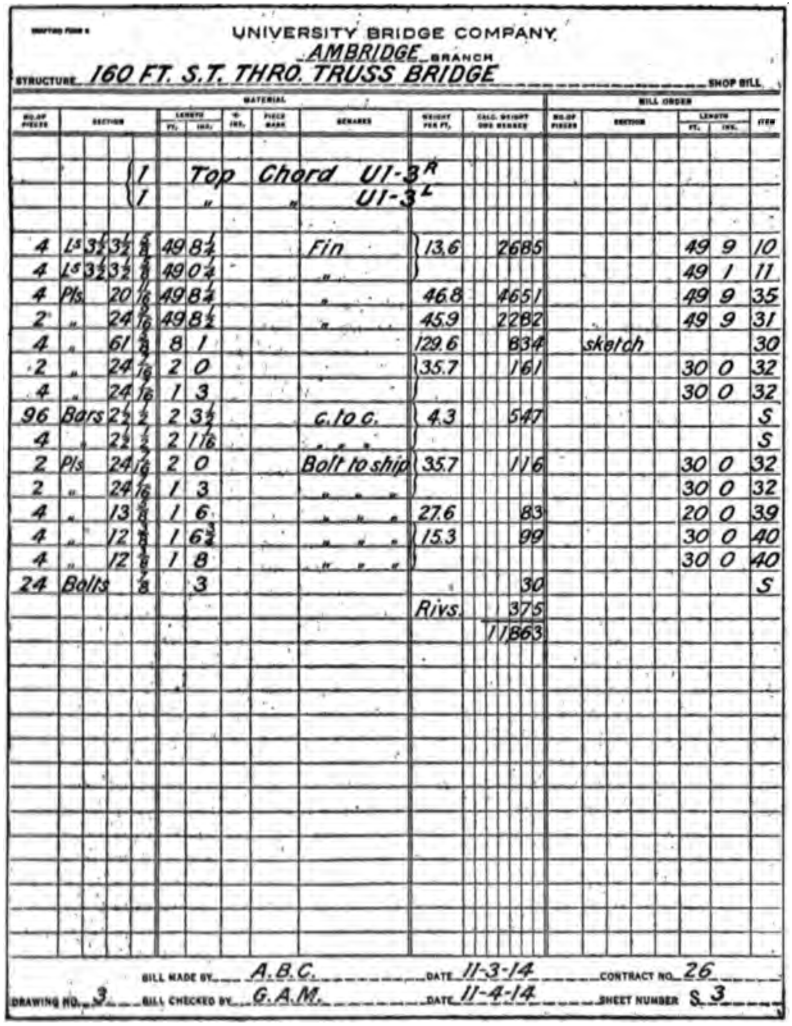
SYNOPSIS: Lists of the component parts of all members of a structure are summarized on shop bills for use in gathering the proper material together. The completed members are listed on shipping bills for the use of the shipper.

1. **A shop bill** is a detailed list of all the material of which one or more members are composed. Shop bills are made for all material (except shop rivets), which is fabricated in the shop in accordance with the drawings. They do not include lumber, corrugated steel, hardware, or other materials which are shipped without shop work directly from the manufacturers to the site. Shop bills are used primarily by men in the receiving yard or stock yard; these men select the material for each member and deliver it to the structural shop as needed. Much of this material is used as it comes from the rolling mill but some must be recut to shorter lengths according to information given on the shop bills. Incidentally, shop bills are used as check lists in different shops; thus for example, in the templet shop each item is checked off on the shop bill as soon as the corresponding templet is completed.

2. **Form.** — Shop bills may be made on separate sheets, or they may be combined with shipping bills (page 171 : 1), or with drawings and shipping bills (page 174 : 1). When made on separate sheets no shop bill should contain the material for more than one sheet of drawings. This is to simplify the distribution of the proper shop bills with each drawing; in fact, they are sometimes attached to the drawing. These shop bills should be **numbered** to correspond to the drawing number; thus the shop bill for drawing number 3 is marked *S* 3, and for drawing number 6 it is marked *S* 6. If more than one page of shop bills are required for a single sheet of drawings, they should be numbered *a*, *b*, *c*, etc., thus: — *S* 4*a*, *S* 4*b*, *S* 4*c*, are all for the drawings on sheet number 4. Combination bills are numbered consecutively, as explained on page 54:5. Shop bills should be made with freehand letters, usually directly in ink without preliminary penciling. According to common drafting-room parlance, they are said to be "written" but this should not be interpreted to imply the use of script. Above all, the figures should be carefully made.

3. A shop bill is divided into **three parts.** The first part is the summary of material taken from the drawing; this part is confined to the left half of the shop-bill form, including the "Remarks" (Fig. 168). This part of the shop bill is usually written by boys or young men who are beginning their careers in the drafting room. For efficiency these billers often work in a separate squad in charge of a Chief Biller. A bill may be checked by another member of the squad, but more satisfactory results are usually obtained if each shop bill is checked by the draftsman who made the corresponding drawing; this is especially desirable if the drawing is complex. The signatures at the bottom of the shop bill are for the men who make and check this first part. The second part of a shop bill is the itemizing, as explained on page 169 : 1. These two parts must be completed before the bills are printed for the shop. The third part is the calculation of the weights of members, as explained on page 170 : 1. The weights are not usually computed unless the contract is let on a pound price basis.

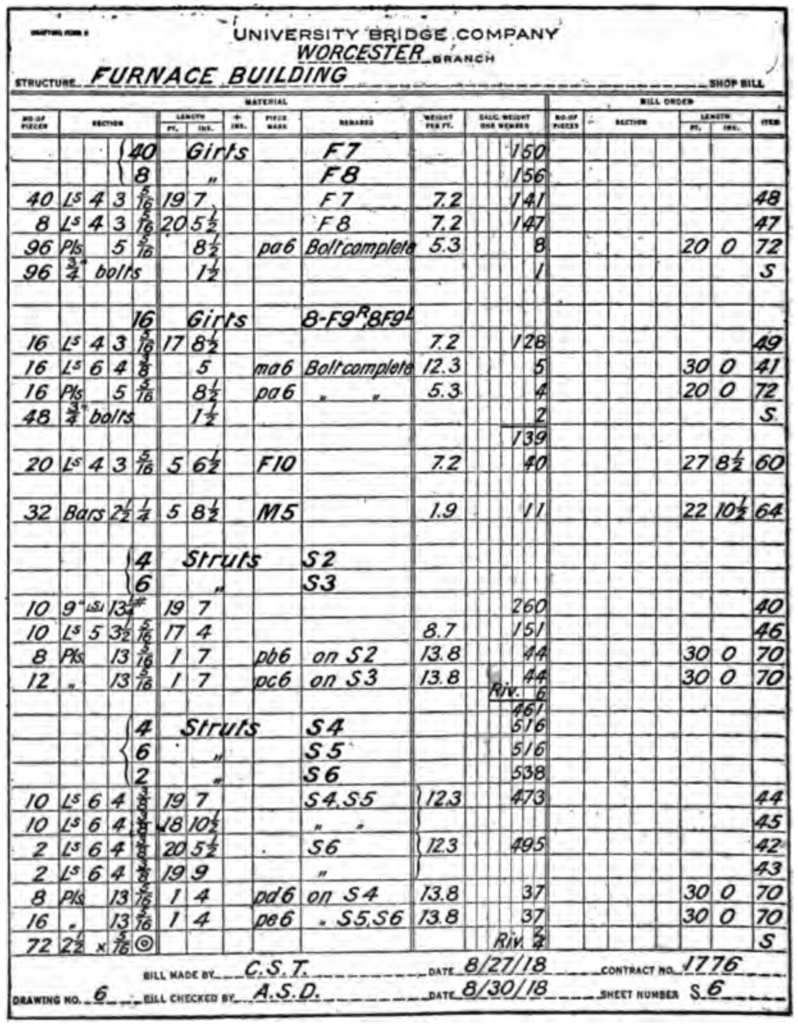
4. **Arrangement.** — In the first part of a shop bill should be listed *all* the material required in making *all* the members which are detailed together, whether or not the members bear the same mark. Thus, in Fig.



UNIVERSITY BRIDGE COMPANY
AMBRIDGE BRANCH
STRUCTURE 160 FT. S.T. THRO. TRUSS BRIDGE SHOP BILL

MATERIAL: No. of Pieces	Section	Length Ft.	Length Ins.	+ Ins.	Piece Mark	Remarks	Weight per Ft.	Calc. Weight One Member	BILL ORDER: No. of Pieces	Section	Length Ft.	Length Ins.	Item
	{1	Top			Chord	U1-3^R							
	1	"			"	U1-3^L							
4	Ls 3½ 3½ 3/8	49	8¼			Fin	13.6	2685			49	9	10
4	Ls 3½ 3½ 3/8	49	0¼			"					49	1	11
4	Pls 20 7/16	49	8¼			"	46.8	4651			49	9	35
2	" 24 7/16	49	8½			"	45.9	2282			49	9	31
4	" 61 3/8	8	1				129.6	834		sketch			30
2	" 24 7/16	2	0				35.7	161			30	0	32
4	" 24 7/16	1	3								30	0	32
96	Bars 2½ ½	2	3½			c. to c.	4.3	547					S
4	" 2½ ½	2	1 1/16			" " "							S
2	Pls 24 7/16	2	0			Bolt to ship	35.7	116			30	0	32
2	" 24 7/16	1	3			" " "					30	0	32
4	" 13 3/8	1	6			" " "	27.6	83			20	0	39
4	" 12 3/8	1	6¾			" " "	15.3	99			30	0	40
4	" 12 3/8	1	8			" " "					30	0	40
24	Bolts 7/8		3					30					S
							Rivs.	375					
								11863					

BILL MADE BY A.B.C. DATE 11-3-14 CONTRACT NO. 26
DRAWING NO. 3 BILL CHECKED BY G.A.M. DATE 11-4-14 SHEET NUMBER S. 3



UNIVERSITY BRIDGE COMPANY
WORCESTER BRANCH
STRUCTURE FURNACE BUILDING SHOP BILL

MATERIAL: No. of Pieces	Section	Length Ft.	Length Ins.	+ Ins.	Piece Mark	Remarks	Weight per Ft.	Calc. Weight One Member	BILL ORDER: No. of Pieces	Section	Length Ft.	Length Ins.	Item
	{40	Girts				F7		150					
	8	"				F8		156					
40	Ls 4 3 5/16	19	7			F7	7.2	141					48
8	Ls 4 3 5/16	20	5½			F8	7.2	147					47
96	Pls 5 5/16		8½		pa6	Bolt complete	5.3	8			20	0	72
96	¾" bolts		1½					1					S
	16	Girts				8-F9^R, 8F9^L							
16	Ls 4 3 5/16	17	8½				7.2	128					49
16	Ls 6 4 3/8		5		ma6	Bolt complete	12.3	5			30	0	41
16	Pls 5 5/16		8½		pa6	" "	5.3	4			20	0	72
48	¾" bolts		1½					2					S
								139					
20	Ls 4 3 5/16	5	6½		F10		7.2	40			27	8½	60
32	Bars 2½ ¼	5	8½		M5		1.9	11			22	10½	64
	{4	Struts				S2							
	6	"				S3							
10	9" [s 13¼#	19	7					260					40
10	Ls 5 3½ 5/16	17	4				8.7	151					46
8	Pls 13 5/16	1	7		pb6	on S2	13.8	44			30	0	70
12	" 13 5/16	1	7		pc6	on S3	13.8	44			30	0	70
								Riv. 6					
								461					
	{4	Struts				S4		516					
	6	"				S5		516					
	2	"				S6		538					
10	Ls 6 4 3/8	19	7			S4, S5	12.3	473					44
10	Ls 6 4 3/8	18	10½			" "							45
2	Ls 6 4 3/8	20	5½			S6	12.3	495					42
2	Ls 6 4 3/8	19	9			"							43
8	Pls 13 5/16	1	4		pd6	on S4	13.8	37			30	0	70
16	" 13 5/16	1	4		pe6	" S5, S6	13.8	37			30	0	70
72	2½ × 5/16 ⊙							Riv. 2/4					S

BILL MADE BY C.S.T. DATE 8/27/18 CONTRACT NO. 1776
DRAWING NO. 6 BILL CHECKED BY A.S.D. DATE 8/30/18 SHEET NUMBER S. 6

Fig. 168. Typical Shop Bills.

168 is included all the material required in making the two chord members of Fig. 124. When only part of a member is shown on the drawing all notes which affect the bill of material must be carefully considered. Thus for a member marked "Symmetrical about the center line" much of the material billed on the drawing should be doubled. When more than one drawing is made on a sheet the members should be grouped on the shop bill in the same way they are grouped on the drawing, as illustrated in Fig. 168, which shows the shop bill for the girts and struts of Fig. 147. One or more blank lines should be left to separate the groups and to provide a space for the total weight where necessary (page 170 : 1). The number, the name, and the mark of each different member should be printed prominently at the head of each group without regard to the vertical lines. Usually only one mark is put on a line, as F 7 and F 8, or S 4, S 5 and S 6 (Fig. 168), although two may sometimes be combined as $F\ 9^R$ and $F\ 9^L$, if it is desired to save space. Members which are composed of single pieces may be billed on a single line, as F 10 or M 5 (compare page 173 : 1). Fine vertical lines are printed under "sections" as an aid to uniformity in billing. With these lines it is unnecessary to use crosses. All four columns are used for angles but the second one need not be used for plates. For beams the sign (#) may be placed in the last column. The material should be listed systematically, beginning with the main material. The details should be grouped as on the drawing, beginning at the bottom or left hand end of a member and proceeding toward the other end. The washers and bolts for any member should be the last items to be billed. Each item which is not common to all the members of a group should be so noted, as in Fig. 168. Similarly, notes should appear opposite each item which is "Milled" or "Finished" (abbreviated "Fin"), "Bolted complete," or "Bolted for Shipment." Assembling marks should be recorded in a special column headed "Piece Mark." Permanent bolts are usually listed on the drawing, hence they are billed as a matter of course. Temporary bolts used to hold loose pieces in position during shipment are usually of odd lengths picked up in the shop; these are not always listed on the shop bill, but they should be so listed for a "pound price" contract. It is usually sufficient to bill all temporary bolts as 3 inches long; in this way an average weight is provided for without undue investigation of details which would be inconsistent with the usual shop practice. The column headed "+ ins." is used only when the material is ordered directly from the shop bills instead of from preliminary order bills, as for example, when shop bills are written for drawings made by a consulting engineer. In this case the complete shop bills can be prepared as quickly as preliminary bills; the latter may be dispensed with and much duplication may be thus avoided. The use of this column on the shop bill is then the same as on the material order bill (page 164 : 6).

1. **Itemizing.** — The second part of the shop bill shows the ordered material from which the yard men should select the steel for the different component parts of a member. All material which is ordered specially for a contract has the contract number and an item number painted on the steel. This item number appears on the preliminary material order bill (page 163 : 2) and it is transferred to the shop bill to indicate the proper assignment of material. The term "Itemizing" is applied to the preparation of this part of the shop bill. The best results are obtained if one man itemizes all of the shop bills of a single contract, or at least of a definite portion of it; this man should be conversant with the practice of the Order Department. Opposite each component part of a member should be placed the **item number** of the material from which it is taken, and suitable notation should be made on the material order bill so that the same pieces will not be used again. The other columns under "Mill Order" are not necessarily used in all cases. In general the **number of pieces** need not be given unless the material is to be ordered from the shop bill instead of from a material order bill, in which case the order department takes care of the whole Mill Order and no itemizing is done in the drafting room. The **section** need be entered only when it differs from that listed in the main part of the bill. For example, an angle which is to be cut from another size should be so noted on the drawing, as 1 $\angle$ $5\frac{1}{2} \times 3 \times \frac{3}{8} \times 1'$-2 (cut from 6×3), illustrated in the bracket on D 13, Fig. 140; in the main part of the shop bill it should be billed $5\frac{1}{2} \times 3 \times \frac{3}{8}$ and the original biller should record $6 \times 3 \times \frac{3}{8}$ in the corresponding place under the Mill Order. The itemizer then provides for the $6 \times 3 \times \frac{3}{8}$ angle. Similarly, plates may be cut from other sizes. Sometimes small plates are changed so that the length becomes the width, as for example, a plate billed 6 ×

$\frac{1}{2} \times 1'$-0 may be itemized as a$\cdot 12 \times \frac{1}{2} \times 6$ plate. Plates which are ordered as sketch plates (page 165 : 2), should be noted as in Fig. 168. The **length** need be given only when it differs from the length billed in the main part of the bill. This may differ by only a fraction of an inch, as in material to be milled (page 165 : 1), or by a large amount when ordered in multiple lengths (page 165 : 2). On account of different grouping on the shop bill and the order bill the number of pieces will often differ. Sometimes more than one item number will have to be placed opposite a single entry on the shop bill, the number taken from each being recorded under "No. of Pieces." More frequently the number of pieces listed on the shop bill will be less than the number on the order bill; care should be taken to make the proper record on the order bill so that the same pieces will not be reassigned. Beam separators or other materials which are assembled in the shop with the beams or other members, should be billed with them in accordance with the drawings. When these materials are made from combination bills they should be itemized on the latter; to save duplication they should not be itemized on the shop bill but reference should be made to the combination bill, as in Fig. 172. All material that is not provided for cn the order bill should be itemized by the order department. Presumably this material will be taken from the plant stock, because care should be taken to write advance orders for all material which cannot be found in stock. Material taken from stock has no item number, but it is indicated by the capital letter S instead. No length need be recorded for stock items, but the section should be indicated if it differs from the billed section.

1. When a contract is based upon a certain price per pound, the payments are determined by the **calculated weight.** The scaled weights are used as an approximate check. For each contract the method of computation should be agreed upon, but undue refinements should be avoided. Usually the weights are determined from the material summarized on the main part of the shop bills; to these are added the weights of the *heads* of the shop rivets. Thus no deduction is made for corners cut off (except for plates ordered by sketch), nor for holes for field rivets or pins; the holes for shop rivets are filled by the rivet shanks so that the weight is not affected. The weights are expressed to the nearest pound; when members are composed of a large number of component parts it may be necessary to carry the partial weights to one decimal. The weights are recorded in ink on the shipping bills, each weight being for a *single* member. The shipper weighs each car load and compares the weight with the sum of the calculated weights of the corresponding members; since the number of members shipped on one car seldom equals the total number of members which are alike, the weight of a single member is more useful than the total weight would be. The shop bills are used for convenience in determining the weights of members; these weights are ultimately transferred to the shipping bills. The weights on the shop bills are used simply as a means toward this end, and for this reason it is unnecessary to ink them; in fact all necessary prints of the shop bills may be made before the weights are added. Usually the weights are computed and checked by young men who specialize in that work, although the draftsmen are sometimes asked to help. Most companies have exhaustive tables to facilitate this work. In computing the weight of one member, the weight of the corresponding number of component parts should be found. Thus for each girt, F 7 or F 8 (Fig. 168), there are two plates and two bolts; the weights should be entered accordingly. Usually the first step in computing weights is to determine from the tables the weights per foot for different sections. These weights may be recorded in the column provided for that purpose; the weights of I-beams and channels are given in the main part of the shop bill so they need not be repeated. The weight of each item is the product of three factors, viz: the weight per foot, the length in feet, and the number of pieces per member. The order in which these factors are multiplied depends upon the method used and upon the factors themselves; it may be more convenient to multiply the length by one of the other factors before the inches and fractions are converted to decimals of a foot. The weights should be placed in the spaces provided for them; fine vertical lines are printed on the forms to simplify the alignment of figures for totaling. When all members of a group weigh the same, a line can be drawn under the weight of the last component part and a single total can be recorded in the blank space between groups, as for F 9. When the weights of the members of a group differ, each must have its own total; to save space

each total may be recorded opposite the mark of the proper member, as for *F* 7 and *F* 8 or for *S* 4, *S* 5 and *S* 6. Care must be taken to include in these totals only the weights of the component parts which form the corresponding members. The weights of the rivet heads should not be overlooked. Since they do not appear on the shop bill they must be counted from the drawings and their weights must be entered upon the shop bill, as in Fig. 168. In counting shop rivets special attention should be given to symmetrical notes, and to group spacing; each rivet has two heads. The weights of rivets and bolts are given on page 304. The weight of a bolt or a rivet may be obtained by the proper combination of the weights of the head, the nut, and the shank of proper length. The weights of standard connection angles for beams include the weights of the shop rivet heads.

1. **A shipping bill** is a summary of members in the form in which they are to be shipped. It is made primarily as a check list for the shipper. All members which bear the same shipping mark should be billed on the same line, but different marks should be listed separately. In each case the number, the name, the mark, and the shipping dimensions should be given; the weight is given only when required (page 170 : 1). The sheet number of the drawing is also recorded. A portion of the bill is printed black so that white spaces will be left on the blue prints for the convenience of the shipper in making records of shipments. The shipping dimensions are usually given to the nearest inch (or half-inch). The extreme "box dimensions" are given, i.e., the three dimensions of a box which would contain the member. The length is given in feet and inches but the other two dimensions at right angles to the length and to each other are expressed in inches, the larger of the two being given first. In case these dimensions do not fairly represent a large member, a freehand sketch with auxiliary dimensions may be drawn in the column headed "Remarks," as shown in Fig. 171. Members should be listed preferably in about the order in which they will be erected. When part of a structure is to be completed before the rest is begun, the material should be billed and shipped accordingly; the shipping bills should be marked with the shipment, as for example, "First Shipment." The last page for any shipment should be marked "Complete." Shipping bills are numbered consecutively, usually with

UNIVERSITY BRIDGE COMPANY
AMBRIDGE BRANCH
STRUCTURE 160 FT. S.T. THRU. TRUSS BRIDGE SHIPPING BILL

Member				Description					Shipments		
No. of Pieces	Name	Mark	Sh't No.	Extreme Dimensions Inches	Length Ft.	Length Ins.	Remarks	Calc. Weight One Member	No. of Pieces	Date	Scale Weight
4	Top Laterals	T1	6	20 × 8	28	11		1331			
4	" "	T2	"	21 × 10	15	$10\frac{1}{2}$		743			
4	" "	T3	"	20 × 8	13	$9\frac{1}{2}$		647			
2	Plates	$P4^R$	"	35 × $\frac{1}{2}$	2	$1\frac{1}{2}$	Bent	126			
2	"	$P4^L$	"	35 × $\frac{1}{2}$	2	$1\frac{1}{2}$	"	126			
2	"	$P5^R$	"	20 × 12	3	0		137			
2	"	$P5^L$	"	20 × 12	3	0		137			
6	"	P6	"	14 × $\frac{3}{8}$	2	$7\frac{1}{2}$		47			
6	"	$P7^R$	"	14 × 4	1	4		30			
6	"	$P7^L$	"	14 × 4	1	4		30			
3	Sway Braces	SB1	"	72 × $10\frac{1}{2}$	14	9	132"	1452			
2	Portals	PS1	"	96 × $12\frac{1}{2}$	18	0	138"	2721			
4	Posts	L1-U1	5	16 × $14\frac{1}{2}$	28	5		2066			
4	"	L2-U2	"	15 × 13	28	$4\frac{1}{2}$		2071			
4	Diagonals	L2-U1	"	14 × $12\frac{1}{2}$	36	$6\frac{1}{2}$		4225			

BILL MADE BY F. J. B. DATE 12/16/14 CONTRACT NO. 26
BILL CHECKED BY M. J. B. DATE 12/18/14 SHEET NUMBER R 3

Fig. 171. Typical Shipping Bill.

a distinctive letter as *R* 1, *R* 2, etc. Bills for different shipments may be numbered in different series as *R* 101, *R* 102, etc., for 1st shipment and *R* 201, *R* 202, etc., for 2nd shipment.

UNIVERSITY BRIDGE COMPANY

YALE BRANCH

STRUCTURE LOFT BUILDING - 2ND FLOOR BEAMS SHOP AND SHIPPING BILL

MEMBER				MATERIAL							MILL ORDERS					SHIPMENTS		
No. of Pieces	Name	Mark	Sh't No.	No. of Pieces	Section	Length Ft.	Length Ins.	+ Ins.	Weight per Ft.	Calc. Weight One Member	No. of Pc's	Sect.	Length Ft.	Length Ins.	Item	No. of Pc's	Date	Scale Weight
2	Beams	B7	B3															
1	"	B8	"															
1	"	B9	"															
				4	10" Is 25#	16	$7\frac{1}{2}$			416					8			
				8	Ls 6 4 $\frac{3}{8}$		$5\frac{1}{2}$			13					S			
										429								
6	Beams	B10	B3															
4	"	B11	"															
				10	15" Is 42#	19	10			833					7			
				20	Ls $2\frac{1}{2}$ $2\frac{1}{2}$ $\frac{1}{4}$	8	$2\frac{1}{2}$		4.1	151			18	5	12			
				20	Ls $2\frac{1}{2}$ $2\frac{1}{2}$ $\frac{1}{4}$	10	$2\frac{1}{2}$						18	5	12			
				40	Ls 4 4 $\frac{7}{16}$		$11\frac{1}{2}$			46					S			
										Riv. 3								
										1033								
6	Beams	B12	B3	6	10" Is 25#	10	11			273					9			

BILL MADE BY C. W. DATE 8/25/14 CONTRACT NO. 6693

BILL CHECKED BY W. B. F. DATE 8/30/14 SHEET NUMBER SR 2

UNIVERSITY BRIDGE COMPANY

YALE BRANCH

STRUCTURE OFFICE BUILDING - 2nd AND 3rd FLOOR BEAMS SHOP AND SHIPPING BILL

MEMBER				MATERIAL							MILL ORDERS					SHIPMENTS		
No. of Pieces	Name	Mark	Sh't No.	No. of Pieces	Section	Length Ft.	Length Ins.	+ Ins.	Weight per Ft.	Calc. Weight One Member	No. of Pc's	Sect.	Length Ft.	Length Ins.	Item	No. of Pc's	Date	Scale Weight
3	Beams	B16	B5	3	12" Is $31\frac{1}{2}$#	15	5			485					3			
				12	Ls 4 4 $\frac{7}{16}$		$8\frac{1}{2}$			34					S			
										519								
4	Beams	C17	B6	4	15" Is 42#	15	0			630					1			
10	BeamGirds	G19	B6	20	6" Is $12\frac{1}{4}$#	9	0			221					5			
				30	1" G.P. Seps.		$6\frac{3}{4}$			3	See page C1							
				30	$\frac{3}{4}$" bolts		8			5					S			
										229								
1	BeamGirds	G20	B6	2	12" Is 40#	14	0			1120					2			
				2	Pls 14 $\frac{5}{8}$	14	0		29.7	832					15			
										1952								

BILL MADE BY A. B. C. DATE 8/17/14 CONTRACT NO. 1700

BILL CHECKED BY V. C. DATE 8/20/14 SHEET NUMBER SR 3

Fig. 172. Typical Shop and Shipping Bills.

CHAPTER XXVII

SHOP BILLS AND SHIPPING BILLS

1. **Shop and Shipping Bills** are often combined on the same form, as in Fig. 172. This is done when the shop bill for each member of a drawing occupies only a few lines, i.e., when each member is composed of but few different parts. These simple shop bills may be arranged to give the necessary information to the shipper so that special shipping bills need not be prepared. It would not be feasible to use combined forms for complex members because the shipping data would be obscured by the details, and the shipper would be burdened with an unnecessarily large number of pages. Usually the combined shop and shipping bills are used for beam work and other simple work drawn on small sheets or printed forms (except those mentioned in the following paragraph); they are sometimes used for simple members drawn on large sheets. For example, the girts and struts of Fig. 147 might be billed on such a form instead of as in Fig. 168. In the combined form the number, the name, and the mark are listed in columns as in shipping bills, except that they are grouped as on the drawing. The remainder of the bill, except the shipper's record, is similar to the shop bill form. The bill of material may be started on the same line as the mark, unless members which bear different marks are grouped together; in this case, it is better to begin on the line below the last mark to avoid the confusion that might arise if part of the material were billed opposite one of the marks. Blank lines should be left between groups. In some work such as structures intended for export, the extreme shipping dimensions are given opposite each mark, just above the detailed list of material. The billed length of a beam is the length which appears on the drawing along with the depth and the weight — usually the ordered length.

2. Shop and shipping bills are sometimes combined with the drawing, as illustrated in Fig. 175 (*a*) and (*b*). The bill portion of these **combination sheets** is much the same as for shop and shipping bills explained in the preceding paragraph.

CHAPTER XXVIII

MISCELLANEOUS DRAWINGS AND LISTS

SYNOPSIS: Some of the more simple drawings may be combined with the corresponding shop and shipping bills on the same sheet; the use of these combination sheets is illustrated. The method of listing field rivets and bolts is explained also.

1. Much of the **miscellaneous material** used in steel construction is of such a nature that comparatively little information need be given on the drawings. This may be because there is little or no shop work to be done, or because most of the shop work is to be carried out according to fixed standards which need not be duplicated on the drawings. Such material is usually drawn on a combination sheet of the general form shown in Fig. 175 (*a*) or (*b*). The use of such forms* reduces the number of sheets and saves duplication, because a working drawing, a shop bill, and a shipping bill are all contained on the same sheet. These sheets are used for rods, anchor bolts and washers, bearing plates and anchors, rollers, crane stops, castings, forgings, etc. Standard castings such as beveled or "O.G." washers, separators, or rail clamps may be listed without a drawing if reference is made to a standard pattern number. When beam separators are to be assembled in the shop and shipped with beams in the form of beam girders, a note to that effect should be placed on the preliminary bill or combination sheet from which they are made so they will not be shipped separately. Reference to this sheet should be made on the shop bill, as shown in Fig. 172. Castings and forgings should be listed on separate sheets for they are made in different departments.

2. **Special printed forms** are used by many companies to simplify the drafting. Such forms are commonly used for crane rails, eye bars, loop rods, pins, clevises, turnbuckles, corrugated steel, etc. A general drawing of one of these pieces is printed on each form with blank dimensions to be filled in. The forms may be so arranged that the different variables may be recorded in tabular form; in this way similar pieces in any contract may be listed upon the same sheet conveniently.

3. The use of a combination sheet with a comparatively large space for the drawing is shown in Fig. 175 (*a*) which illustrates a typical **cast-iron base** † for an office-building column. The tabular portion is arranged as a combination shop and shipping bill (page 173 : 1); not all of the blanks are required for castings. Note that the tops of these cast bases are finished to furnish uniform bearing for the columns. Holes are drilled in the tops for the bolts which hold the columns in place during erection; it would be impracticable to punch holes or to drive rivets in cast iron because of the danger of cracking the casting. Cored holes (cast in place) are sometimes used for bolts, but they should be made $\frac{1}{8}$ inch larger than the bolts to allow for irregularities in the casting. Holes are cored in the bottoms of large cast bases to permit the more even distribution of grout which is poured after the bases are in place.

4. The use of a combination sheet arranged for a large number of items with comparatively small drawing space is shown in Fig. 175 (*b*). This illustrates the **rods** for the mill building shown in Fig. 156. It is perhaps necessary to show more complete drawings for some rods, and

* See footnote, page 83

† For a table of dimensions of the American Bridge Company's standard cast-iron bases, see Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

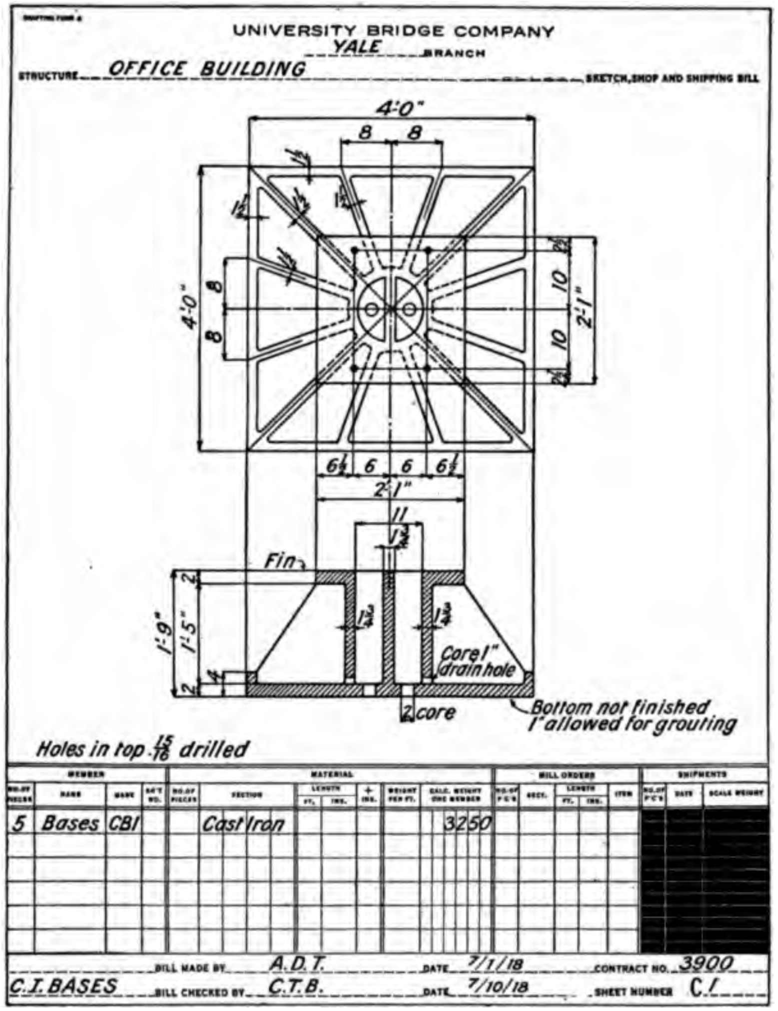


Fig. 175 (*a*). Combination Sheet with Drawing.

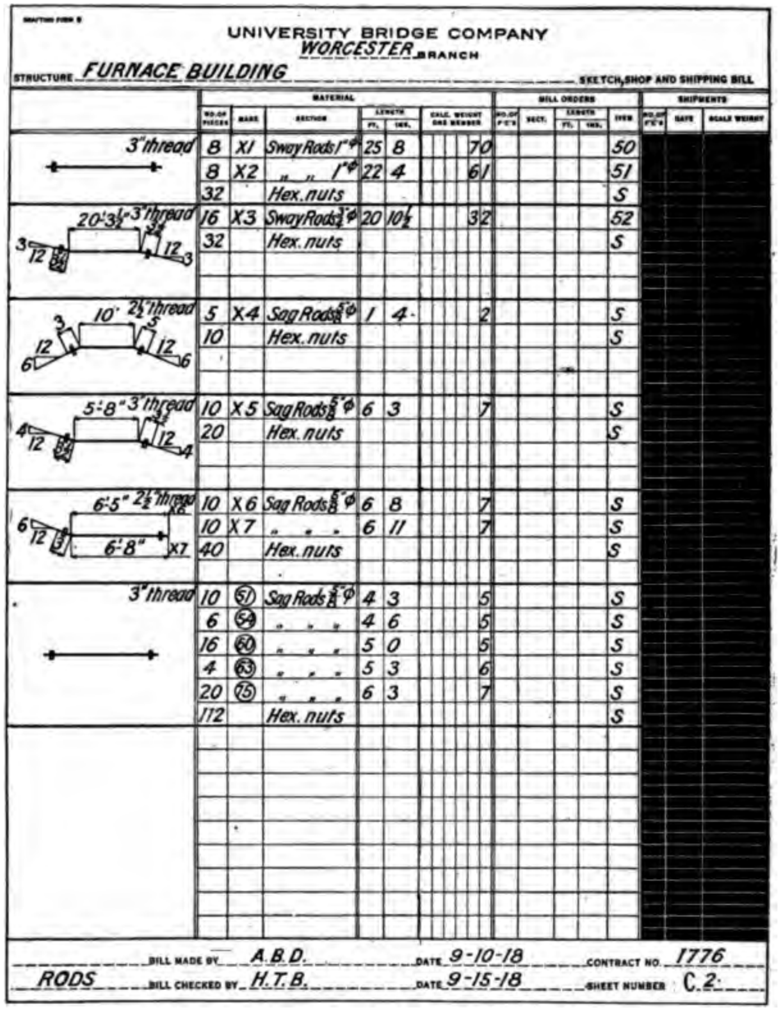


Fig. 175 (*b*). Combination Sheet with Small Sketches.

some companies require them for all rods; but usually sufficient information can be given if the rods are shown conventionally by single lines, as in the figure. It is assumed that dimensions are taken along the center lines. Main diagonal rods are marked in the usual manner with a letter X followed by a specific number (page 80 : 7). It is impractical to paint the mark on each individual sag rod or tie rod; they are

UNIVERSITY BRIDGE COMPANY
WORCESTER BRANCH
STRUCTURE FURNACE BUILDING ERECTOR'S LIST

NO. OF PIECES	DIAM.	RIVETS		BOLTS LENGTH	HEAD	NUT	WASHERS	GRIP	MEMBERS CONNECTED
60	3/4"	2 1/2						1 1/16	K1 to Cols
40	"	3						1 7/16	" " "
120	7/8"	2 1/4						3/4	" " Girders
112	3/4"			1 1/2	Hex.	Hex.		11/16	F7, F8, F9 to Cols
112	"			2	"	"		1	" " " " "
32	"			1 3/4	"	"		3/4	F9 to Cols.
40	"			1 1/2	"	"		9/16	F10 to Purlins
64	"			1 1/2	"	"		9/16	M5 to Girts.
60	"			1 1/2	"	"		11/16	S2, S3 to Cols.
60	"			1 3/4	"	"		15/16	" " " "
96	"			1 1/2	"	"		11/16	S4, S5, S6 to Cols.
48	"			2	"	"		1 1/16	" " " " " "

BILL MADE BY E.A.P. DATE 9/14/18 CONTRACT NO. 1776
BILL CHECKED BY J.K.L. DATE 9/20/18 SHEET NUMBER EF2

Fig. 176. Erector's List of Field Rivets and Bolts.

usually shipped in bundles and the bundles are marked. In case rods become mixed before they are used, the erector must identify them by direct measurement. If a rod is indicated on the erection diagram by an X and a number, the erector must refer to a list of rods to determine the proper length; this extra step may be avoided except in the case of bent rods, if all straight tie rods and sag rods are marked according to their lengths. Thus, on the diagram the length in inches of one rod in each panel may be inscribed in a circle, as shown in Fig. 156 or Fig. 158; the circle is used to distinguish these marks from others. Since the tie rods or sag rods of a given structure are usually of the same diameter, the erector simply needs to know the required lengths; it is more convenient for him to find the lengths where the rods are shown on the diagrams, than to have to refer to a separate list. The lengths of tie rods and sag rods are usually made in multiples of 3″ in order to reduce the number of different sizes; in determining the lengths, use may be made of the dimensions on page 316. For the weights of rods see page 315.

1. A summary of **field rivets and bolts** must be prepared for each contract in order that the proper number of each size may be shipped to the site for use in erection. Before this summary can be made, a detailed "erector's list" must be prepared to show the number and the size of the rivets to be used in each connection. To have each connection provided for once, and only once, it is well to have one man list the rivets for a whole structure or for a definite portion of it; he should work systematically to avoid duplication. Perhaps the best plan is to take one sheet at a time, and to list only those rivets by which each member on that sheet is connected to the *supporting* members. The rivets for all members which bear the same shipping mark should be listed together, and the rivets for similar members may be combined whenever this can be done conveniently without adding to the burdens of the erector.

2. A typical **erector's list** of field rivets and bolts is shown in Fig. 176; this provides for the connection of the knee braces in Fig. 140 and of all the members in Fig. 147 to the columns of Figs. 135 and 137. The erection diagram of Fig. 156 should be used as a guide in determining the relative positions of the members. A rivet is made with one head in place. The shank should be long enough to extend through the parts to be connected far enough to provide sufficient metal for the forma-

tion of the second head and for the upsetting of the shank to fill the enlarged hole (page 30 : 4). Rivets commonly used in structural work are either "button head" or "countersunk" (page 40 : 6); both heads may be alike, or one may be button and the other countersunk. The length of a button head rivet is measured from the under side of the head; the length of a countersunk rivet is the extreme length overall. The "grip" of a rivet is the total thickness of metal through which it must pass, i.e., the sum of the thicknesses of the parts connected. The length of a rivet is the sum of the grip and the extra length required for upsetting. The thicknesses of I-beam and channel flanges at the rivet lines are shown in the tables on pages 298 to 302. The lengths of rivets vary slightly according to the shapes of the heads adopted as standard by different companies. Tables are used to determine the proper lengths of rivets for different grips.* These tables are usually arranged for grips varying by eighths of an inch. The grips for both rivets and bolts are usually recorded to the nearest sixteenth, but this is unnecessary unless one is likely to be substituted for the other; for rivets, grips in sixteenths should be increased $\frac{1}{16}$ before the lengths are found. Care should be taken to differentiate between countersunk and button head rivets; more metal is required to form a button head than a countersunk head. It is well to record the lengths of all countersunk rivets as soon as the grips are recorded, being careful to place them in the proper column. It is usually more convenient to omit the lengths of button head rivets until one or more pages of the erector's list are otherwise complete, when they can all be recorded. The bolts are listed upon the same form as the rivets, but the style of head (button, square, or hexagonal) and the style of nut (square or hexagonal) should be indicated as shown. The length of a bolt is measured from the under side of the head; it should be a multiple of $\frac{1}{4}$ inch. A bolt should extend from $\frac{1}{16}$ to $\frac{1}{4}$ inch beyond the nut to insure the full bearing of all the threads in the nut. The thickness of a nut is the same as the diameter of the bolt with which it is used. When washers are used, a bolt should be made correspondingly longer.

* The standards of the American Bridge Company are given in the "Pocket Companion" of the Carnegie Steel Company, and in Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

UNIVERSITY BRIDGE COMPANY
WORCESTER BRANCH
STRUCTURE FURNACE BUILDING FIELD RIVETS AND BOLTS

NO. OF PIECES	DIAM.	RIVETS		BOLTS				REMARKS	CALC. WEIGHT TOTAL	SHIPMENTS		
				LENGTH	HEAD	NUT	WASHERS					
75	$\frac{3}{4}$	$2\frac{1}{2}$							34			
60	"	3							32			
140	$\frac{7}{8}$	$2\frac{1}{4}$							87			
400	$\frac{3}{4}$			$1\frac{1}{2}$	Hex.	Hex.			282			
110	"			$1\frac{3}{4}$	"	"			81			
170	"			2	"	"			132			
				Erection Bolts.								
40	$\frac{3}{4}$			3	Hex.	Hex.	$2\frac{1}{4} \times \frac{3}{8}$	2 washers each	65			
40	$\frac{7}{8}$			$2\frac{1}{2}$	"	"	$2\frac{1}{2} \times \frac{3}{8}$	" " "	81			

BILL MADE BY E.A.P. DATE 9/14/18 CONTRACT NO. 7776
BILL CHECKED BY J.K.L. DATE 9/20/18 SHEET NUMBER C 4

Fig. 177. Summary of Field Rivets and Bolts.

1. A **summary** of field rivets and bolts is made on a form similar to the erector's list except that blanks for weights and shipments are sub-

stituted for the grip and the list of members connected, as shown in Fig. 177. All the field rivets and bolts on a whole contract may be summarized in one list unless different portions of a structure are grouped under different shipments, when the rivets may be divided accordingly. The number of each different size and style of rivet and bolt may be determined by means of a sheet ruled into columns; each item from the erector's lists may be recorded in the proper column and then each column may be totaled. These totals should be increased by certain percentages in order to allow for waste, for miscounts, and for misplacements. These percentages should be greater for small numbers in order to provide a safe margin, as indicated in the following table: *

Total number required.	Percentage to be added.	
	Rivets.	Bolts.
1 to 10	100	50
11 to 50	50	25
51 to 100	25	12½
101 to 500	15	7½
over 500	10	5

In the illustration, the summary has been made simply for the rivets and bolts shown in the typical erector's list and not for the whole contract. The weights of rivets and bolts should be computed only when the weight of the other steel work is required. The weight recorded is the total weight of all the rivets or bolts listed on one line. The weights of rivets, bolts, and washers are tabulated in many handbooks and structural company standards. The tables on page 304 give the weights of the shanks, the heads, and the nuts; the weight of the shank can be multiplied by the proper length and combined with the weight of the head or the head and nut. . The weight of a washer is approximately three-tenths of the product of the thickness in inches by the net area in square inches; the holes are usually ⅛ inch larger than the bolts.

* Standards of the American Bridge Company.

1. **Erection Bolts.** — As soon as a member is swung into the proper position in a structure it is held in place by erection bolts put in some of the holes of the field connections; after the remaining holes are riveted these bolts are replaced by rivets. In this way the erection gang is enabled to make greater progress for they do not have to wait for the rivets to be driven; special gangs of riveters drive the rivets. If permanent bolts are to be used, the erection gang may use permanent bolts for erection so that they need not be replaced; these permanent bolts are provided in suitable lengths as explained above. The temporary fitting-up or erection bolts are made longer than permanent bolts and they are provided with washers. The washers make the bolts more serviceable because a bolt may be used for different grips by varying the number of washers; incidentally the use of washers facilitates the removal of the bolts. Not all structures are erected by the company that furnishes the material, and erection bolts should not be furnished unless specified. Each bolt should be provided with two ⅜-inch washers, and the lengths should vary by ½ inch. The number should be based upon the number of field rivets furnished. For buildings and miscellaneous structures the number for each grip should be about 30 per cent of the number of field rivets; for bridges this should be increased to 50 per cent, or even 80 per cent for the floor systems and the tension members where more bolts are likely to be lost.

CHAPTER XXIX

CHECKING AND CORRECTING DRAWINGS

SYNOPSIS: In this chapter are given points for the checker to consider while checking a drawing. These points should also be anticipated by the detailer in order that he may guard against mistakes.

1. **A checker** is a person who "checks" or approves the correct portions of a drawing and indicates the mistakes for correction. After these mistakes have been verified and corrected by the detailer, the checker ascertains that all corrections have been made and then signs the drawing to show that he has assumed responsibility for every detail, every figure, and every note, in short for the entire drawing. The checker is usually a man of greater experience than the detailer, and no draftsman is allowed to check the work of others until he is able to make drawings which are comparatively free from errors.

2. **Not all Drawings are Completely Checked.**— Some structural companies have fixed policies regarding checking, while other companies leave to the chief draftsman or the squad foreman in charge of a given contract the decision of whether or not the drawings for that contract shall be checked. The importance of careful checking depends upon the ability and the skill of the draftsman; it is essential for the drawings made by inexperienced men. The importance increases with the size of a contract and the number of draftsmen engaged upon it, because of greater difficulty in insuring the proper agreement between the drawings of connecting members. A checker can work to better advantage than the detailer because the drawings of connecting members are usually farther advanced so that he can compare the completed connections more definitely. The majority of companies have all their drawings checked because they are unwilling to assume the risk of serious mistakes which might be made if only one draftsman were responsible. Since no draftsman is infallible these companies believe that every drawing should be the product of two minds. Some companies find that it is economical to have all drawings made by experienced draftsmen whose work is comparatively free from errors without being checked; these companies maintain that occasional mistakes can be rectified at a smaller cost and to better advantage than all drawings can be checked, particularly for structures which are to be erected within comparatively short distances from the plant. Perhaps a better method would be to have the main dimensions, the strength of the connections, and the spacing of all holes for field connections checked, but not the spacing of the shop rivets or the sizes of component parts which will necessarily be verified in the shop. Sometimes a "field check" of an entire contract is made either instead of or in addition to the usual checking. Such a check is made after most of the drawings are ready for the shop but before any of the members are shipped, and preferably before the shopwork has progressed very far. The object of this check is to insure the proper erection of the structure, and it includes the checking of all main dimensions and all field connections; it should preferably be made by one person. A field check should be unnecessary for small contracts, but it may be desirable for large contracts (*a*) when the number of field connections is relatively large, as in export work; (*b*) when the cost of field changes would be prohibitive on account of inaccessibility or the lack of facilities; or (*c*) when the drawings of

connecting members have been made by a large number of detailers and checked by several checkers all working simultaneously.

1. **A detailer** should become familiar with the work of a checker and apply the checker's methods to his own work as far as practical. This is especially true if his work is not to be checked by another person, but no drawing should be submitted to a checker until the detailer feels confident that it is comparatively free from error. He should never allow a mistake of which he is aware to go to the checker uncorrected. A beginner is judged not so much by the mistakes he makes as by the mistakes he repeats. A man who profits by each correction which is brought to his attention and avoids making similar mistakes on future drawings will soon surpass the men who are not so careful. The detailer will find that many of the suggestions given in this chapter may be applied to his drawing to advantage before it is submitted to the checker. Many of these checks may be made at a glance, and they should always be applied. The detailer should also determine each important dimension in more than one way if possible. He should never divide a distance into rivet spacing or other subdivisions without adding all the dimensions and comparing the sum with the proper total.

2. **A checker** should work systematically so that he will not miss any portion of the drawing. He should collect all data concerning the member or members shown on a given drawing, and make sure that nothing is overlooked. It is not sufficient to check what the draftsman has done, but it is especially important to discover his errors of omission. A good checker is distinguished from a poor one very largely by his ability to detect omissions. The checker should check the most important parts of a drawing first, proceeding logically so that no part will be checked until the parts upon which it is dependent are checked.

3. The following **suggestions** show the more general points to be considered in checking a drawing, arranged approximately in logical order: —

(1) *General Appearance:* see that the general arrangement is satisfactory; that the proper views are used to show the member to the best advantage; that all views bear proper relations to one another; that the proper scale is used so that the drawing is clear and not crowded; and that not too many different members are combined on the same drawing thus making it too complicated.

(2) *Main Dimensions:* see that the proper type of connection is provided at the supports; that the proper clearances are allowed for erection; that the main dimensions are given conspicuously in such a manner that they will be of most service; and that the main material is billed conventionally in accordance with the main dimensions and with the design.

(3) *Connections at Supports:* see that the strength of the rivets and the component material is sufficient to carry the stresses; that the size and the spacing of the holes for the field rivets correspond to those in the supporting members; and that the material is properly billed.

(4) *Other Connections:* see that provision is made for all connecting members; that the connections are properly located; that the number, the size, and the spacing of the holes for the field rivets correspond exactly to those in the connecting members; that all connecting members can be erected without interference; and that all material is properly billed.

(5) *Lattice Bars and Rivets:* see that all lattice bars, batten plates, and rivets are properly spaced to conform to the specifications and to the usual rules; that all rivets and bolts are so located that they can be put into position; that sufficient clearance is allowed, if possible, so that the rivets can be driven by machine; that rivets which should be countersunk or flattened are properly indicated; and that necessary stitch rivets are provided.

(6) *Edge Distances:* see that all necessary edge distances are given, as for example, those which will prevent interference with erection.

(7) *Special Cutting:* see that sufficient dimensions and notes are given for special cutting or coping, particularly when a variation will cause interference or mar the appearance of the finished structure.

(8) *Dimensions:* see that interdependent dimensions bear the proper relation to one another; that the distance between any two points is the same when obtained by adding the intervening dimensions on any one of two or more parallel dimension lines; and that no dimension lines or other lines or points appear to coincide on the drawing unless it is proper that they should coincide.

(9) *Milling:* see that all surfaces to be faced, milled, or planed are properly marked.

(10) *Notes:* see that all notes are given which render the drawing clearer or more definite, such as "symmetrical about the center line"; that all differences between members which are combined on the same drawing are clearly noted; and that all loose material which is not to be shipped separately is noted "Bolted for shipment".

(11) *Shipping Marks:* see that each member is provided with the proper shipping mark; that the marks correspond to the marks on the erection diagrams; that proper distinction is made between rights and lefts; and that the required number of members is correct.

(12) *Sizes of Rivets:* see that the sizes of all rivets, bolts, holes, and washers are given either in a general note or in special notes.

1. **Indicating Mistakes.** — The checker should indicate all mistakes so that the draftsman will be able to make the necessary changes without disturbing the checker's notes. See Fig. 181. These notes should be preserved until the checker has convinced himself that all changes have been made correctly. The mistakes may be indicated either on the tracing or on a blueprint. The former method is more direct and more common; it is also quicker because the checker does not have to wait for a print to be made, and the draftsman does not need to refer to a print before making each change on the tracing. The blueprint method is used when a permanent record of the checker's marks is desired.

The temporary mark of the checker may be made on the tracing with either blue or soft black pencil. It is easier to maintain a sharper point on the black pencil than on the blue, and hence the figures and notes need not be made so large. The black marks may be removed more readily than the blue. Marks on blueprints may be made satisfactorily with either black or yellow pencils. Red pencils should not be used on either tracing cloth or blue prints. The red marks cannot be erased easily from the cloth — in fact it is impossible to remove some of them. There are certain combinations of red crayon on blue prints which cause unnecessary eye strain on account of an optical illusion which makes it difficult for the eye to focus on the red lines.

A freehand ring or loop drawn around a dimension, a note, or a whole detail usually signifies that the enclosed portion is incorrect. A line may be drawn from the ring to a convenient clear space on the drawing where the correct value or other note may be written by the checker so that it need not be disturbed by the draftsman when he makes that or any other correction. An inked ring is often drawn around a group of holes with a line leading to a special note (page 52 : 4), but in order to avoid ambiguity such a ring should not include figures or notes. A penciled ring may be drawn around a detail which is likely to be changed; such a ring should invite suspicion or doubt, and no use should be made of figures or other data taken without verification from within the ring. Accordingly it is not good practice to encircle any dimension or

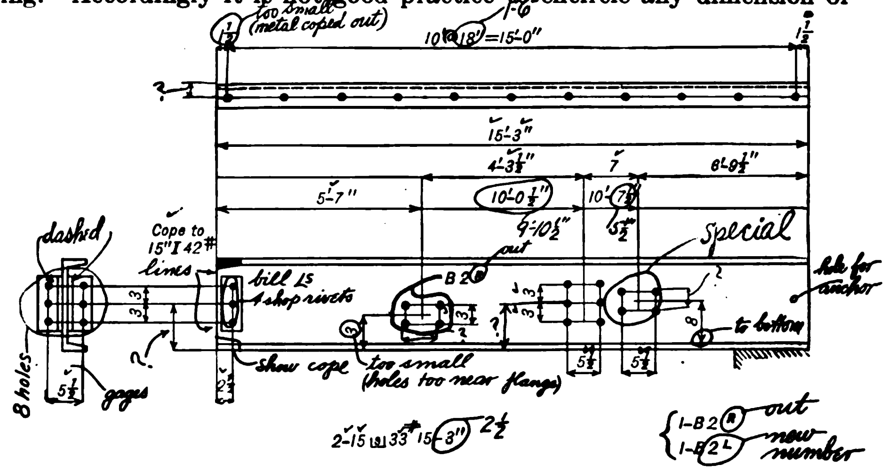


Fig. 181. Method of Indicating Mistakes.

note unless it is either wrong or doubtful because of incomplete or revised information.

2. **Check Marks.** In checking a drawing a checker usually places a dot or a v-shaped check mark over each correct dimension or note (Fig. 181). These check marks indicate his progress and make him less liable to overlook parts of the drawing; they are of special value on complicated drawings. Check marks may be made with a pencil and later erased, or they may be made permanently in red or blue ink for future record. If ink is used, some checkers place the check marks underneath corrected figures to distinguish them from those which were originally correct.

1. **Back-checking.** — A draftsman should never change a drawing according to the checker's notes until he is convinced that such changes should be made. Unless he "back-checks" or verifies each correction before making it, certain parts of the drawing are left unchecked, the value being the checker's instead of the detailer's. No checker is infallible, and every sheet that is supposed to be checked should be completely checked. A checker usually indicates the values which he believes to be correct; this is the simplest method for him to use. It is also the most convenient method for the detailer who back-checks conscientiously. Some checkers wish to insure back-checking by withholding the correct values until after the draftsman has supplied them. These checkers indicate mistakes for the draftsman but record the correct values on a blueprint or in a book for their own use in checking the corrected drawing. For convenience in identification they may number the mistakes consecutively.

2. In **checking students' drawings** it is well for the instructor to indicate the mistakes so that the students will have to determine the correct values themselves before they can correct the drawings. A practical method which has proven successful in the author's classes is to indicate not the correct values, but the reference numbers to paragraphs which will suggest the nature of the mistakes. This may result in more work for the instructor but not as much more as might be imagined. The mistakes on the similar drawings of a whole class which are corrected at any one time cover only a limited range; most of the mistakes are made by several men so the paragraph numbers are soon memorized. The method may be simplified by spreading the drawings out on a large table and checking one connection or other small portion of all the drawings before another portion is checked. The extra burden on the instructor seems justified by the benefit to the student. He gets a much better understanding of each correction and he is less liable to repeat the same mistake. His increased familiarity with the text makes the book more valuable for future reference. It is good practice for the students to check each other's drawings and one question on a test or examination may well be devoted to the correction of a drawing which contains many mistakes. In this work a student's grade should be based upon his net score, the number of mistakes he makes being deducted from the number which he detects.

3. **Corrections.** — After the draftsman has verified all corrections he should carefully erase the incorrect portions of his drawing and replace them with the correct. The figures should not be crossed out and new ones should not be superimposed as in the *unchecked* field notes of surveying. There is no necessity for preserving the incorrect figures of the draftsman after the sheet has been checked and both the checker and the draftsman are convinced that the figures are wrong. If these figures were retained, the appearance of the drawing would be marred, the drawing would be made less distinct, and the draftsman's mistakes would be exposed to all who use the drawing. It is important that all changes be made without disturbing the checker's marks. If any note of the checker is erased accidentally the draftsman should pencil a conspicuous question mark near the corresponding change to draw the attention of the checker to the change. The checker may simply check the changes and then assume responsibility for the drawing by signing his initials to the sheet. If his marks were erased he would have to recheck much or all of the drawing before he could be sure that it was correct, particularly if much time had elapsed since he first checked the drawing.

4. **Revisions.** — If changes are made after prints have been issued, the prints must be revised or replaced by new prints. In either case the changes should be indicated conspicuously. If the old prints are revised with black ink or colored crayon the changes are apparent. If new prints are issued the revised portions should be underscored or otherwise marked with colored crayon to attract attention.

PART III — THE DESIGN OF DETAILS

CHAPTER XXX

SHEAR AND BENDING MOMENT

SYNOPSIS: Every structural draftsman should know how to design a beam or a girder to satisfy different requirements. The design is comparatively simple after the shear and the bending moment are known. The first step is to find the shear and the bending moment under different conditions.

1. **The terms** "shear" and "bending moment" are convenient expressions for certain quantities which are used repeatedly in the design of beams, girders, and similar members. Their definition presupposes the knowledge of forces and moments. Shear and bending moment depend upon the magnitudes and relative positions of *external* forces, such as the applied loads and the reactions at the supports. Since the majority of external forces for which beams are designed are either horizontal or vertical, it is convenient to resolve all other forces into horizontal and vertical components. Any other system of rectangular coördinate axes may be chosen if preferred, as, for example, when inclined beams are subjected to normal forces.

2. **Principles.** — In Statics a system of forces in equilibrium must satisfy three conditions, viz.: the algebraic sum of the horizontal components of all the forces must equal zero; the algebraic sum of the vertical components of all the forces must equal zero; and the algebraic sum of the moments of all the forces about any point of moments must equal zero. These three equations of equilibrium are commonly indicated thus: $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$. In this book they will be referred to as the "H equation," the "V equation," and the "M equation," respectively.

3. **Signs.** — In order to obtain algebraic sums it is necessary to adopt a system of signs for components and moments. Any system may be used provided that system is followed consistently throughout a given problem. It is convenient to adhere to one system in all problems in order to avoid confusion and to eliminate one source of error. In this book all horizontal components toward the *right*, all vertical components *upward*, and all moments of forces which tend to cause *clockwise* rotation about the point of moments are considered *positive*. Conversely, components toward the left or downward, and moments of forces which tend to cause counterclockwise rotation are considered negative.

4. **Forces Considered.** — The equations of equilibrium may be applied to a beam as a whole, or to a portion of a beam. When the whole beam is considered, all the external forces which act on the beam must be in equilibrium. Thus, any unknown external force, such as the reaction at a support, is found by applying the principles of equilibrium to the beam as a whole. When only a portion of a beam is considered, the external forces acting upon that portion are not, as a rule, in equilibrium, but they are held in equilibrium by the internal forces or stresses which act in the beam. For convenience, the external forces on a portion or segment of a beam are first considered and quantities termed "shear" or "bending moment" are obtained; similar expressions are found from the internal forces and the two combined must satisfy the equations of equilibrium. The consideration of the internal forces depends upon the

form of the beam and the material; this is a matter of design and is treated in Chapters XXXI and XXXIII, pages 197 and 218.

1. **The "shear"** *on a segment of a beam is the algebraic sum of all the external forces which act upon that segment.* It is not a force but a *sum* of forces; it is not the sum of all the external forces on the beam (which would be zero) but the sum of those forces which are on the segment. This sum cannot be considered to act at any one point or at any one cross section, although the internal forces which act at a given cross section must be sufficient to resist this sum or shear. It is better to speak of "the shear *for* a section taken at a given point" meaning the shear on a segment of the beam made by this section. Such a section should not coincide with the line of action of any force, but it may be considered to pass infinitely close to the force on either side so that the whole force is either on one segment or the other. For any section, the shear on one segment is numerically equal to the shear on the other segment but with opposite sign; this is necessarily so because the algebraic sum of the two must equal zero to satisfy the V equation of equilibrium applied to the whole beam.

2. **The "bending moment"** *at any point of a beam is the algebraic sum of the moments of all the external forces at the left of that point, the latter being taken as the point of moments.* A cross section at the point of moments cuts the beam into two segments, only one of which is considered in finding the bending moment. The bending moment found from the forces on the left-hand segment is numerically equal to the bending moment found from the forces on the right-hand segment but opposite in sign; thus the algebraic sum of the two equals zero to satisfy the M equation of equilibrium applied to the whole beam. It is less confusing for the beginner always to use the left-hand segment; he is less liable to use the wrong signs. It is of course simpler to use the segment which has the smaller number of forces, but until the student has become proficient it is better for him to use the left segment. In case the right-hand segment contains a smaller number of forces the whole beam may be turned around; that is, it may be considered to be viewed from the opposite side when the sketch is drawn. It is imperative that the units of bending moments be expressed in every case. Bending moments in pound-feet and in pound-inches are both used so extensively that failure to distinguish between them is the cause of many serious mistakes. In this book M_B and m_B are used to indicate bending moments in pound-feet and pound-inches, respectively.

3. The use of **formulas** for finding shear or bending moment should be reduced to a minimum. The fundamental principles of shear and bending moment are so easily applied to beams under usual conditions that it is useless to attempt to memorize or even recognize the many formulas so often used for beams under different forms of loading. It is unnecessary to differentiate between simple and cantilever beams (page 83 : 1) since the same methods are applicable. The extensive use of formulas makes a man either dependent upon his memory to a dangerous degree or else a slave to a handbook. A student who constantly reverts to a fundamental principle as simple as "the algebraic sum of the forces (or the moments) on the left of a given section" in finding shear or bending moment will probably never forget how to solve a similar problem without a handbook. Furthermore, he can solve the problems more surely and about as quickly in this way as if he substituted in formulas which cannot long be retained in the memory. Formulas are serviceable in the solution of beams with fixed ends or continuous beams with more than two supports because the underlying theory is too complex to be developed for every problem. Certain short cuts will suggest themselves to the man who has a large number of similar problems to solve, but for the most part they should be avoided by the student. In no case should a man use a short-cut formula unless he can readily derive it.

4. **Sketches.** — For every beam a single-line sketch should be drawn upon which are shown all external forces, including the loads and the resultant reactions at the supports. The magnitude and direction of each known force should be recorded, as well as all dimensions necessary to locate each point of application. The remaining magnitudes and directions should be recorded as soon as determined. The reactions of a simple beam are considered to act at the centers of the supports, and the effective length is the distance from center to center of supports. A beam which is supported at one end only must have that end imbedded in a wall or otherwise fixed. The reaction is not a single force, for obviously no single force applied at one end could hold the beam in equilibrium. At least two forces are required, either parallel or non-parallel. If the

end is imbedded in a wall it is difficult to know just how the forces act. Inasmuch as both the shear and the bending moment can be found without the reactions being known it is better to draw a sketch similar to Fig. 187 (*a*) without indicating any forces at the support. At all events the reaction should not be indicated as a single force. The sketch of the whole beam should be used only when the beam is considered as a whole, as in finding reactions. A separate sketch should be drawn in determining each shear and each bending moment. Each of these sketches should show only the segment of the beam with the forces which are considered in finding any one shear or bending moment. A sketch used for shear need show only the direction and the magnitude of each force on the segment; a sketch used for bending moment should in addition show the distances between the forces. Such a sketch should show clearly the forces which are to be considered in any one step. The time required in drawing such a sketch is offset by the time saved in mentally selecting the proper forces for each step and eliminating others, and the results obtained are much more reliable. The efficacy of drawing a separate sketch for each segment has been proven unquestionably by observations taken for many years in the author's classes; the large majority of all mistakes in shear and bending moment were due to the omission of such sketches, particularly in problems which involved uniformly distributed loads.

1. Some systematic **arrangement** should be adopted for all computations. The result of each step should be placed so that it may be readily found; its significance should be apparent; and the method of solution should be indicated. The arrangement adopted in this book * has been found entirely satisfactory in many classes. First a sketch is drawn to show the forces acting on the whole beam. The computation may be arranged under different headings if it seems desirable. The results of different steps with the proper units are tabulated at the left of the page so that the equality signs which follow the results are in a vertical column. If a vertical line is printed or drawn near the left of the sheet the results and units may be placed at the left of this line, and the equality signs at the right. After each equality sign is the indication of the calculation by which the corresponding result is obtained, followed by a brief description of what the result signifies. In case a separate sketch is required for any step it may be drawn at the right of the description. Thus, in general, each step is completed on one line. It is of great advantage to instructors to have all students arrange their work uniformly. It is of advantage to everyone to arrange his work so that he or anyone else can readily find the result of any step, what that result means, and how it was obtained. These results are more conspicuous if tabulated at one side of the page instead of being mixed in with other work. The advantage of having the results at the left is to leave no space between the results and the corresponding indications of the computation; this would be difficult to effect if the results were placed at the right, because one cannot anticipate how much space the computation and the description will require.

2. **Reactions.** — *Concentrated loads.* The first step in finding either the shear or the bending moment is to determine all the external forces, or at least all that will be needed. No reactions need be found for beams which are supported at one end only, because the shear and the bending moment may be determined without them. For a system of parallel forces there cannot be more than two unknowns. Usually the lines of action of all the forces, and the magnitude of all but one or two are known; however the two unknowns may be the line of action of one force (including the point of application) and the magnitude of one force, not necessarily the same force. The unknowns are commonly the magnitude (and direction) of the reactions at the supports. The beam is treated as a whole and all the external forces are considered to be in equilibrium. Any inclined forces may be resolved into components so that the horizontal (or other) components are in the same line of action; by taking the points of moments in this line of action the problem becomes similar to the more usual case of a horizontal beam with vertical forces. In the remainder of this chapter only horizontal beams with vertical forces will be considered. The principles may be readily adapted to vertical beams with horizontal forces, or even inclined beams with inclined forces. When there are no horizontal forces there can be no use of the H equation. The V equation cannot be used alone if there are two unknowns. The M equation involves two unknowns unless the point of moments is chosen in the line of action of one of the unknown

* For further advantages see page 208 : 3.

forces. When the point is so chosen the lever arm and hence the moment of this unknown becomes zero, and the magnitude of the other unknown may be determined from a single equation. Similarly, by taking the point of moments in the line of action of the force just found, the second unknown may be found from another M equation. The sign which results from the solution of the M equation is *not* the sign of the force but of its moment. The direction of the force must be found accordingly, with due consideration of the point of moments. This is of special importance in cantilever beams for the direction of the reactions cannot be determined by inspection. The V equation may be used as a check. The second unknown might be found from the V equation after the first unknown is determined, but its accuracy would be dependent upon the accuracy of the first unknown. By using the M equation, each unknown is determined from the original data independently. It is good practice to avoid the use of computed values when original data can be used equally well, because an error in the first value does not affect the second. As a student becomes proficient he may indicate the result and the solution for each reaction upon a single line, as on page 192 : 1 and subsequent pages. He should also be on the lookout for symmetrical loads for which each of two reactions is equal to one-half the sum of the loads.

1. The three **illustrative problems** which follow show how reactions, shears, and bending moments may be found for typical beams with *concentrated loads.*

FIRST PROBLEM — SIMPLE BEAM

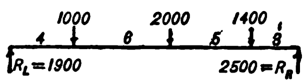


Fig. 186 (*a*).

To find the reactions (beam treated as a whole):

$0 = R_L \times 18 - 1000 \times 14 - 2000 \times 8 - 1400 \times 3$ (point of moments at R_R)

$+1{,}900\# = R_L$ (clockwise means upward)

$0 = 1000 \times 4 + 2000 \times 10 + 1400 \times 15 + R_R \times 18$ (point of moments at R_L)

$-2{,}500\# = R_R$ (counterclockwise means upward)

$0 = 1900 - 1000 - 2000 - 1400 + 2500$ = check.

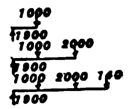
To find the shear:

$+1{,}900\# = R_L$ = shear for a section between R_L and the 1000 load.

$+900\# = +1900 - 1000$ = shear for a section between the 1000 and 2000 loads

$-1{,}100\# = +1900 - 1000 - 2000$ = shear for a section between the 2000 and 1400 loads

$-2{,}500\# = +1900 - 1000 - 2000 - 1400$ = shear for a section between the 1400 load and R_R.



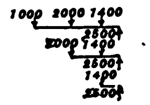
These values may be checked from the right-hand segments as follow (see page 184 : 1):

$-1{,}900\# = -1000 - 2000 - 1400 + 2500$

$-900\# = -2000 - 1400 + 2500$

$+1{,}100\# = -1400 + 2500$

$+2{,}500\# = R_R.$

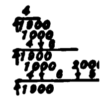


To find the bending moment:

$7{,}600\#\text{ft.} = 1900 \times 4 = M_B$ at the 1000 load

$13{,}000\#\text{ft.} = 1900 \times 10 - 1000 \times 6 = M_B$ at the 2000 load.

$7{,}500\#\text{ft.} = 1900 \times 15 - 1000 \times 11 - 2000 \times 5 = M_B$ at the 1400 loac

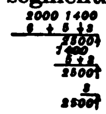


These values may be checked from the right-hand segments as followe

$-7{,}600\#\text{ft.} = 2000 \times 6 + 1400 \times 11 - 2500 \times 14$

$-13{,}000\#\text{ft.} = 1400 \times 5 - 2500 \times 8$

$-7{,}500\#\text{ft.} = -2500 \times 3.$



SECOND PROBLEM — CANTILEVER BEAM

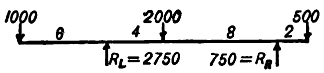


Fig. 186 (*b*).

To find the reactions (beam treated as a whole):

$0 = -1000 \times 18 + R_L \times 12 - 2000 \times 8 + 500 \times 2$ (point of mc ments at R_R)

$+2{,}750\# = R_L$ (clockwise means upward)

$0 = -1000 \times 6 + 2000 \times 4 + R_R \times 12 + 500 \times 14$ (point of m ments at R_L)

$-750\# = R_R$ (counterclockwise means upward)

$0 = -1000 + 2750 - 2000 + 750 - 500$ = check.

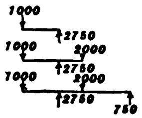
To find the shear:

−1,000# = shear for a section between the 1000 load and R_L

+1,750# = −1000 + 2750 = shear for a section between R_L and the 2000 load

−250# = −1000 + 2750 − 2000 = shear for a section between the 2000 load and R_R

+500# = −1000 + 2750 − 2000 + 750 = shear for a section between R_R and the 500 load.

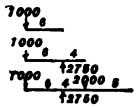


To find the bending moment:

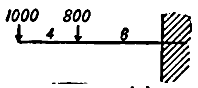
−6,000#ft. = $-1000 \times 6 = M_B$ at R_L

+1,000#ft. = $-1000 \times 10 + 2750 \times 4 = M_B$ at the 2000 load.

−1,000#ft. = $-1000 \times 18 + 2750 \times 12 - 2000 \times 8 = M_B$ at R_R.



THIRD PROBLEM—CANTILEVER BEAM SUPPORTED AT ONE END ONLY

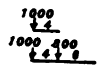


Reactions not required. Fig. 187 (*a*).

To find the shear:

−1,000# = shear for a section between the 1000 and 800 loads

−1,800# = −1000 − 800 = shear for a section between the 800 load and the wall



To find the bending moment:

−4,000#ft. = $-1000 \times 4 = M_B$ at the 800 load

−14,800#ft. = $-1000 \times 10 - 800 \times 9 = M_B$ at the face of the wall.

1. **Uniformly Distributed Loads.** — Some loads are distributed uniformly along a beam instead of applied at points of concentration. The weights of the beams and the weights of superimposed floors are examples of such loads. Certain other loads, such as tracks or crowds of people, are for convenience considered uniformly distributed. The simplest way to treat this form of loading is to make temporary substitutions of equivalent concentrated loads. One substitution may be made in finding the reactions, but a different substitution must be made in finding each different shear or bending moment. Uniformly distributed loads are represented by rectangles, the lower edges of which are the single lines representing the beams. For each step a section should be indicated on the original sketch. In finding a reaction, this section passes through the point of moments; in finding a shear this section is the section for which the shear is taken; in finding a bending moment this section passes through the point where the bending moment is required, i.e., the point of moments. For each different section a separate sketch should be drawn of one segment of the beam (usually the left, page 184 : 2) showing clearly the portion of the uniformly distributed load on that segment. A second sketch should be drawn in which this portion of the uniformly distributed load is replaced by a single equivalent concentrated load applied at the center of gravity of *this portion* of the uniform load on the segment. These equivalent loads should never be shown on the same sketch with the uniform loads because they do not both act at the same time. It is well to indicate these equivalent loads by forces below the beam line to distinguish them from actual concentrated loads which are superimposed. The importance of drawing these separate sketches should not be overlooked. A very common mistake is to draw one sketch for finding a reaction and to use the same sketch for finding a shear or a moment; this gives entirely different results. The sketch used in finding a shear may be used in finding a bending moment, provided the section is taken at the same point.

2. The following **illustrative problems** show how reactions, shears, and bending moments may be found for typical beams with *uniformly distributed loads*. More commonly these loads extend the full length of the beam (see next paragraph); but the same method is applicable.

FIRST PROBLEM — SIMPLE BEAM

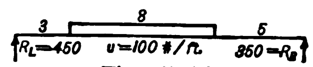


Fig. 187 (*b*).

To find the reactions (beam treated as a whole):

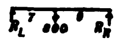
0 = $R_L \times 16 - 800 \times 9$ (point of moments at R_R

+450# = R_L (clockwise means upward)

0 = $800 \times 7 + R_R \times 16$ (point of moments at R_L)

−350# = R_R (counter clockwise means upward)

0 = +450 − 800 + 350 = check.



To find the shear:

$-50\# = 450 - 500 =$ shear for a section midway between the supports. 

To find the bending moment:

$+2{,}350\#\text{ft.} = 450 \times 8 - 500 \times 2.5 = M_B$ midway between the supports.

SECOND PROBLEM — CANTILEVER BEAM

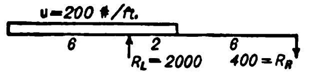
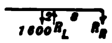


Fig. 188 (*a*).

To find the reactions (beam treated as a whole):

$0 = -1600 \times 10 + R_L \times 8$ (point of moments at R_R)

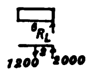
$+2{,}000\# = R_L$ (clockwise means upward)

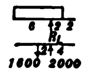
$0 = -1600 \times 2 + R_R \times 8$ (point of moments at R_L) 

$+400\# = R_R$ (clockwise means *downward*)

$0 = -1600 + 2000 - 400 =$ check.

To find the shear:

$+800\# = -1200 + 2000 =$ shear for a section just to the right of R_L. 

$+400\# = -1600 + 2000 =$ shear for a section midway between R_L and R_R. 

To find the bending moment:

$-3{,}600\#\text{ft.} = -1200 \times 3 = M_B$ at R_L

$-1{,}600\#\text{ft.} = -1600 \times 6 + 2000 \times 4 = M_B$ midway between R_L and R_R.

1. **Combined Loads.** — Every beam must support the uniformly distributed load due to the weight of the beam itself. Besides this it may support an additional uniformly distributed load, a system of concentrated loads, or both. All uniformly distributed loads may be combined before the shear or bending moment is computed, but it is usually better to treat the concentrated load systems separately. This is especially true when the concentrated loads are variable or movable, or when the short-cut method of the following paragraph can be used to advantage. The total shear for any section is the sum of the shear due to the uniformly distributed load and the shear due to the concentrated loads. The total bending moment at any point is the sum of the bending moment due to the uniformly distributed load and the bending moment due to the concentrated loads. Shears should only be combined when they are for the same section. Bending moments should, as a rule, be combined only when they occur at the same point (compare page 192 : 2). Whether the shear or bending moment is to be found for all or for only part of the loads on a beam it is important that the reactions used correspond to the loads under consideration.

2. **Short-Cut Rule.** — It seems desirable to introduce one "short-cut" method for finding the bending moment at any point in a *simple* beam (page 83 : 1) under a uniformly distributed load which extends the full distance between the two supports. This form of loading occurs so frequently in practice that the use of such a method is justified. It should be used, however, only by those who have thoroughly mastered the more general method and who can readily derive the rule for themselves; otherwise the rule is likely to be misused with regret. The rule is as follows: *The bending moment at any point of a simple beam with a full length load uniformly distributed is equal to one-half the unit load multiplied by the product of the segments.* It is important to realize that this method is not general, but that it is limited to a beam with a *full load* uniformly distributed between the supports with *no load* extending beyond either support. It does *not* therefore apply to concentrated loads, to partial uniform loads, or to cantilever beams. The unit load is usually in pounds per linear foot, and the lengths of the segments cut by the point of moments are usually in feet; the resulting bending moment is thus in pound-feet. To prove the rule let us consider a simple beam of length L feet, with a load of U pounds per foot, as shown in Fig. 188 (*b*). Let it be desired to find the bending moment at a point which is Y feet from R_L. The reactions may be found in the regular way, but it is obvious that for symmetrical loads each reaction is equal to one-half the total load, or in this case $\frac{UL}{2}$. The equivalent concentrated load acting at the center of the Y segment is UY. The bending moment is found in the usual way as follows:

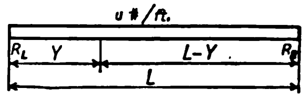
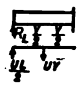


Fig. 188 (*b*).

$$\frac{UL}{2} \times Y - UY \times \frac{Y}{2}$$



or

$$\frac{U}{2}Y(L - Y).*$$

1. **Relation Between Shear and Bending Moment** — *Concentrated Loads.* † — When the bending moment is required at several different points of a beam it is sometimes convenient to obtain one moment from another. *For fixed concentrated loads the bending moment under any load is equal to the bending moment under an adjacent load plus the product of the shear for a section between the loads multiplied by the distance between the loads.* Care must be taken in combining moments to use the correct signs; the signs of the additional products are not necessarily the same as the signs of the shears. From one end of a beam to the other the bending moments will increase as long as the shears bear the same sign, but they will decrease when the shears bear the opposite sign. This is illustrated in the design of pins, page 280 : 3.

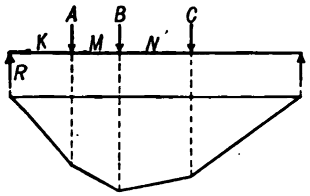


Fig. 189.

The rule stated above will now be proved. In Fig. 189 three different forces A, B, and C are spaced at unequal intervals of K, M, and N, causing a reaction R at the left-hand support. The following bending moments may be obtained in the usual manner:

At R, O
At A, $+RK$
At B, $+R(K + M) - AM$, or $RK + (R - A)M$
At C, $+R(K + M + N) - A(M + N) - BN$, or $RK + (R - A)M + (R - A - B)N$

For any section between R and A the shear is R,
" " " " A " B " " " $R - A$,
" " " " B " C " " " $R - A - B$

It is evident that the bending moment at A is equal to the moment at R plus the shear for a section between R and A multiplied by K. Similarly, the bending moment at the point C is equal to the bending moment at point B plus the shear for a section between B and C multiplied by N. This may be found to apply to either simple or cantilever beams for any number of loads and for any spacing.

2. **Live Loads.** — Not all loads are fixed in position or in magnitude. The loads which may be placed temporarily on beams or girders and which may be changed in position are termed "live loads" or "moving loads" to distinguish them from "dead loads" or "static loads" which remain in one position. The live loads are usually superimposed, as trains, trucks, people, water, etc.; the dead loads include the weights of the beams together with any other dead loads such as tracks, floors, tanks, etc. When the live loads are placed in a certain position and stopped they become static loads, and shears and bending moments may be found in the same manner as for quiescent loads. Beams which are to support live loads must be designed to support them in every possible position, and accordingly it is necessary to place them where they will have the greatest effect. The effects of different types of loads in different positions are analyzed in the following paragraphs. When these live loads are applied suddenly the shears and bending moments are greater than when the loads are stationary. Provision is made for this "**impact**" by adding a certain percentage of the live load according to specifications. Impact will not be considered further in this chapter, but use is made of an impact formula in the illustrative problem on page 225 : 1.

3. **Live-Load Shear — Simple Beams.** — For a given section in a simple beam, the maximum shear will usually occur when the longer segment is fully loaded and the shorter segment is unloaded. In Fig. 190 (a) is shown the position of a uniformly distributed load which will cause the

* For the maximum moment at the center, $Y = L - Y = \frac{L}{2}$, hence $M_B = U\left(\frac{L}{2}\right)^2$. This will be found to be even simpler in application than the common handbook formulas $M_B = \frac{UL^2}{8}$, $M_B = \frac{WL}{8}$, or equivalent expressions, because smaller quantities are involved. The above expression is given for the sake of comparison, but it should not be used as a formula; it is better to use the rule because that applies to the moment at any other point in the beam as well as at the center.

† For uniformly distributed loads see page 244 : 3.

maximum shear for a section at the left end of the load. This shear is equal to the reaction at the end of the unloaded segment because there are no intervening loads to be added algebraically. Note that a new reaction must be calculated for each different position of the load. If the load in the figure were moved back to the right, the left reaction, and therefore the shear, would be reduced. If the load were advanced past the section on to the shorter segment the left reaction would be increased but the shear at the section would be reduced because the additional load on the shorter segment would have to be subtracted from the reaction, and the reaction would be increased by only a fraction of this additional load. This analysis proves that the maximum shear for any section will occur when the load extends throughout the longer segment only. Similarly, a single concentrated load to cause the maximum shear for any section should be placed infinitely close to the section with the load considered to be entirely upon the longer segment. The larger of two concentrated loads should be placed the same as a single load, with the second load as close to it as possible but on the longer segment, as in Fig. 190 (*b*). Three or more concentrated loads are placed in the same manner unless one or more of the end loads are considerably smaller than the intermediate loads. In this case the maximum shear may occur when the first heavy wheel is placed adjacent to the section with the smaller wheel on the shorter segment, as in Fig. 194. This is illustrated by Cooper's engine loads, page 193 : 1. The absolute maximum shear, or the maximum of maxima, for a simple beam is for a section taken infinitely close to the reaction so that the shear is equal to the maximum reaction.

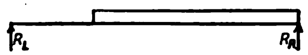


Fig. 190 (*a*).

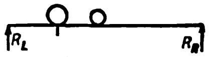


Fig. 190 (*b*).

1. **Live-Load Shear — Cantilever Beams.** — The maximum shear on a beam which is supported at one end only will be for a section taken at the support when the full load is on the beam (regardless of position). Similarly, the maximum negative shear for a cantilever beam which overhangs one or both supports will be for a section taken just outside the reaction when the full load is on the overhanging portion; this shear is not affected by any load between the reactions. The maximum positive shear for the same beam will be for a section just inside the reaction when the portion of the beam which extends outside of that reaction and the portion between the reactions are both loaded, as in Fig. 190 (*c*); there should be no load on any portion of the beam which may extend beyond the other reaction. This may be easily proved. For typical shear diagrams, see Fig. 193.

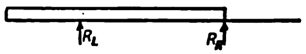


Fig. 190 (*c*).

2. **Live-Load Bending Moment — Simple Beams.** — *Uniformly Distributed Loads.* The maximum bending moment at any point B feet from the left end of a simple beam L feet long for a live load of U pounds per foot uniformly distributed *over a distance C feet* will occur when the end of the load is a distance $X = B - \frac{BC}{L}$ from the left end of the beam. The bending moment at this point will increase as the distance C increases; it is greatest when the load extends the full length L. The absolute *maximum* will occur at the center of the beam under a full load. These statements will now be proved in the order given. Referring to Fig. 190 (*d*), we can find the following values in the usual manner:

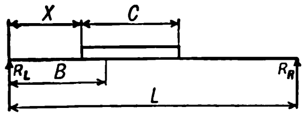


Fig. 190 (*d*).

$$\frac{UC\left(L - X - \frac{C}{2}\right)}{L} = R_L$$

$$\frac{UC\left(L - X - \frac{C}{2}\right)B}{L} - U(B - X)\,\frac{(B - X)}{2} = M_B \text{ at a distance } B \text{ from } R_L.$$

This may be reduced to this form:

$$U\left[BC - \frac{BC^2}{2L} - \frac{B^2}{2} + \left(B - \frac{BC}{L}\right)X - \frac{X^2}{2}\right].$$

The value of X which will produce the maximum bending moment may be found by differentiating this expression with respect to X, and placing the first derivative equal to zero, thus:

$$U\left(B - \frac{BC}{L} - X\right) = 0, \qquad \text{whence} \qquad X = B - \frac{BC}{L}.$$

If we substitute this value of X in the last expression for bending moment we have:

$$U\left[BC - \frac{BC^2}{2L} - \frac{B^2}{2} + \left(B - \frac{BC}{L}\right)^2 - \frac{\left(B - \frac{BC}{L}\right)^2}{2}\right]$$

which may be reduced to this form:

$$UB\left(1 - \frac{B}{L}\right)\left(C - \frac{C^2}{2L}\right).$$

The value of C which will produce the maximum bending moment may be found by differentiating this expression with respect to C, and placing the first derivative equal to zero, thus:

$$UB\left(1 - \frac{B}{L}\right)\left(1 - \frac{C}{L}\right) = 0, \quad \text{whence} \quad C = L.$$

If we substitute this value of C in the last expression for bending moment we have:

$$UB\left(1 - \frac{B}{L}\right)\left(L - \frac{L^2}{2L}\right), \quad \text{or} \quad \frac{UL}{2}\left(B - \frac{B^2}{L}\right). \text{ (Compare page 188 : 2.)}$$

The value of B which will produce the maximum bending moment may be found by differentiating this expression with respect to B, and placing the first derivative equal to zero, thus:

$$\frac{UL}{2}\left(1 - \frac{2B}{L}\right) = 0, \quad \text{whence} \quad B = \frac{L}{2}.$$

1. **Live-Load Bending Moment — Simple Beams.** — *Concentrated Loads. The maximum bending moment at any point of a simple beam for a system of moving concentrated loads will occur when one of the concentrated loads is at that point.* The maximum bending moment due to *two* loads will occur when the larger load is placed at the given point with the other load on the longer segment as near the first load as possible. The placement of more than two loads to give the maximum bending moment is not so simple. Often two or three different loads in turn must be placed at the point, and the corresponding bending moments must be computed and compared. The following criterion must be satisfied by the critical load which is placed at the given point: *the average load on one segment should be greater than the average load on the whole beam when the critical load is considered on that segment, but less when it is considered on the other segment.** The average load is the sum of the loads on a segment, or whole beam, divided by the corresponding length. Often more than one load will satisfy this criterion. It is sometimes possible to tell by inspection that one of two wheels is the critical wheel; in this case it is unnecessary to apply the criterion. The greatest bending moment that can occur on a simple beam for a series of moving concentrated loads will be found *near* the center of the beam, but not as a rule at the center. *The absolute maximum bending moment, or maximum of maxima, will occur under one of the concentrated loads when the center of the beam is midway between that load and the center of gravity of all the loads on the beam.* The critical load is always one of the two loads adjacent to this center of gravity, usually the nearest load. The relative position of the center of gravity will often change when the loads are moved, because one or more loads may come on one end of the beam while others may move off the other end. Enough trials must be made to make sure that the bending moment under one load is greater than under either adjacent load when each is placed for the maximum. For special suggestions for placing the concentrated wheel loads of Cooper's conventional locomotives, see page 195 : 1. To prove that the maximum bending moment on a simple beam will occur when the critical load P (Fig. 191) is placed as far on one side of the center of the beam as the center of gravity of all the loads on the beam is on the other side, let Y represent the distance from P to the center of gravity of ll the loads Q, X the distance from P to the center of gravity of all the loads N on the left segment, and B the distance from P to the reaction R_L. Then $\frac{Q(L - B - Y)}{L} = R_L$, and $\frac{Q(L - B - Y)B}{L} - NX$ = the bending moment under the load P. The value of B which will produce the maximum bending moment may be found by differentiating this expression with respect to B, and placing the first derivative equal to zero, thus:

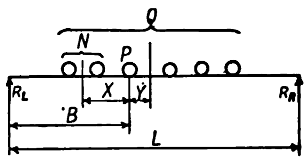


Fig. 191.

* For derivation see Marburg's "Framed Structures and Girders," Part I, McGraw-Hill Book Co., Inc., New York.

$$\frac{Q(L - 2B - Y)}{L} = 0, \quad \text{whence} \quad L - 2B - Y = 0, \quad \text{or} \quad B = \frac{L}{2} - \frac{Y}{2}.$$

1. **Illustrative Problem** — *Two Concentrated Live Loads.* — (See page 195 : 1 for more than two loads.)

To find the maximum bending moment on a 20-foot beam due to two concentrated live loads of 8000# and 6000# respectively, spaced 7 feet apart. The distance from the larger load to the center of gravity may be found by moments, taking the point of moments under the larger load. The sum of the moments of the loads taken separately must equal the moment of the sum of the loads (i.e., the resultant), thus: $6000 \times 7 = (6000 + 8000)\ X$, whence $X = 3$ feet. The 8000 load should be placed 1½ feet from the center of the beam, as shown in Fig. 192 (*a*). The reaction may be found either from the original two loads or from the resultant thus:

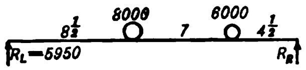


Fig. 192 (*a*).

$$5950\# = (8000 \times 11\tfrac{1}{2} + 6000 \times 4\tfrac{1}{2}) \div 20 = R_L$$

or

$$5950\# = 14{,}000 \times 8\tfrac{1}{2} \div 20 = R_L$$

The maximum bending moment under the larger load is 50,580#ft. $= 5950 \times 8\tfrac{1}{2}$.

2. **Bending Moment — Simple Beams.** — *Combined Loads.* Concentrated loads exist only in conjunction with uniformly distributed loads. The latter may be due simply to the weight of the beam itself, or to track or floor loads as well. The maximum bending moment for the uniformly distributed load is at the center, but the maximum for moving concentrated loads is usually at another point. The maximum for the combined loads will not be found at either of these points but somewhere between them. This may be shown graphically by plotting the moment diagram for the uniformly distributed load above a base line and that for the concentration loads below the line, as in Fig. 192 (*b*). The longest ordinate *m* from one curve to the other represents the maximum total bending moment. The position of this point of moments is in the section for which the shear is zero; this may be found either algebraically or graphically. However, it is seldom necessary to locate this point because at least one of the moment curves is nearly horizontal between the two points of maximum bending moments. It is customary to combine the maximum bending moment due to *moving* concentrated loads with the maximum due to uniformly distributed loads, although they do not occur at the same point. The slight error is on the side of safety. The maximum bending moments due to *fixed* concentrated loads and uniformly distributed loads should not be combined unless they occur at points so close together that the excess will not be too great.

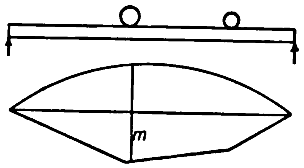


Fig. 192 (*b*).

3. **Live-Load Bending Moment — Cantilever Beams.** — The maximum bending moment for a beam supported at one end only will occur at the edge of the support when the loads are placed as far from the support as possible. For beams which overhang one or both of two supports there will be a maximum positive and a maximum negative bending moment. One will be found at one of the supports when the overhanging end is fully loaded, the loads being placed as far from the support as possible The other maximum for fixed loads will occur at that section between the supports for which the total shear is zero; the maximum for live loads will occur midway between the supports when only the portion of the beam between the supports is loaded, the cantilever end being empty. The point of contraflexure is the point at which the bending moment is zero. Typical shear and moment diagrams are shown in Fig. 193. Positive bending moments have been plotted downward to represent more nearly the direction in which the beams bend.

4. **Restrained beams and continuous beams** are not treated here because the structural draftsman is seldom called upon to design them. Beams and girders with end connection angles are in a measure restrained or "fixed" at the ends. The amount of restraint depends so largely upon the efficiency of rivets in tension and upon the rigidity of the supporting members that it is customary to design such beams as simple beams. Even beams which are inserted into masonry walls are not always sufficiently imbedded to insure a fixed condition, except such beams as lintels, which support masonry walls over openings. It is safer to design these beams as simple beams than to place so much dependence upon the masons. Office-building beams are often so braced to the

columns that they must be designed as fixed beams, but these and similar beams are usually designed in the Designing Department and therefore are beyond the scope of this book.* Beams which have more than two supports are statically indeterminate. Their use is not recommended ordinarily. If desired these continuous beams may be designed by the

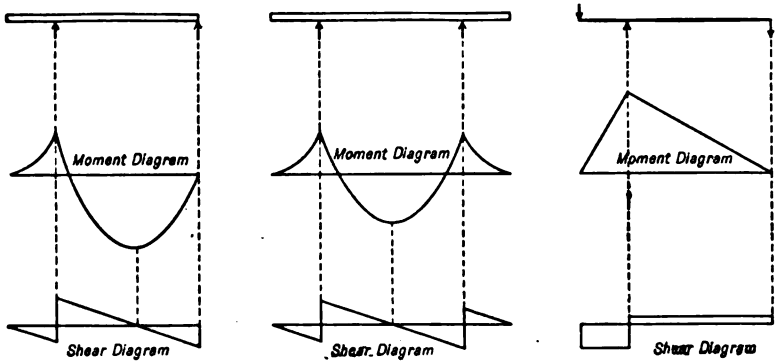


Fig. 193.

" Theorem of Three Moments " *, but the lighter beams (such as purlins) which extend over three supports primarily to give rigidity to a structure are more often designed as simple beams.

1. **Conventional Wheel-Load Systems.** — The floor systems (beams, stringers and floor beams) and the main girders of railroad bridges are usually designed to support a specified live load of two freight locomotives with concentrated wheel loads followed by a uniformly distributed train load. For very short spans an alternate loading of the two driving axles of a passenger locomotive is specified. Formerly almost every railroad specified a different locomotive until Theodore Cooper proposed his system of conventional engine loads for the sake of uniformity. He classified his engines according to the axle loads, the spacing between loads being the same for all engines. As the specified axle loads have increased considerably since his system was adopted, the spacing between loads is less than for actual locomotives. The difference in results is on the side of safety (except for cantilever bridges and turntables) and Cooper's Loadings are quite generally specified. The engines are classified according to the axle loads on the driving wheels, and all other axle loads vary proportionately. The axle loads on the drivers of an " E60 " loading are each 60,000 pounds, and the train is 6000 pounds per linear foot of track. Similarly, the corresponding loads of an " E40 " loading are 40,000 and 4000, and the shears and bending moments for E40 are four-sixths (two-thirds) of the corresponding shears and bending moments for E60. This relation makes it possible to prepare a table of shears and moments for one class of loading and to use this table in getting shears and bending moments not only for that class but, by proportion, for other classes as well. Such a table † is given on page 318. E60 loading is chosen in accord with the more recent specifications of the leading railroad companies. In the table the *axle* loads per *track* have been divided by two to give the *wheel* loads per *rail*, and all shears and moments result accordingly. This usually corresponds to the loads supported by one stringer or girder. When two stringers are used under each rail these values must be subdivided again. Shears and moments are taken as if the loads were applied directly to the stringers or girders, although as a matter of fact the loads are transmitted to them through the track (i.e., the rails and the ties). The distances between wheels have been plotted in the table to a scale of $\frac{1''}{16} = 1'$. The number of wheels which may come on a given span may best be determined by plotting the length of the span along the edge of a strip of paper to this scale and then sliding the strip along the diagram. The significance of the different values is explained in the table. Students should verify enough values for shears, moments, and positions of centers of gravity to enable them to understand their full meaning. The use of the table is illustrated by the solution of the typical problems which follow.

2. **Maximum Shear on Beam** — *Cooper's Loading.* — The loading which will cause the maximum shear on a simple beam depends upon

* See Kirkham's "Structural Engineering," McGraw-Hill Book Co., Inc., New York; Fuller and Johnston's "Strength of Materials," John Wiley and Sons, Inc., New York, and others.

† Numerical values for shears, bending moments, floor-beam reactions, etc., are tabulated in Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

the span. For spans up to 12.5 feet the special loading of two 37,500# wheel loads 7 feet apart should be used. For longer spans (except those between 23.0 and 27.3 feet) the maximum shear on the beam will equal the *left* reaction (when the engines face toward the left as in the diagram) when the first driver (wheel 2) is placed at the *left* end. For spans between 23.0 and 27.3 the maximum shear on the beam will equal the *right* reaction when wheel 5 is placed at the *right* end. This is due to the effect of wheel 1, which in this case is on the beam.

Illustrative Problem — General Case. — To find the maximum shear on a 74-foot deck girder for Cooper's E60 loading. The figures printed along the vertical line under wheel 2 give the distances from that wheel to the wheel over the corresponding vertical of the zigzag line. The distance to wheel 14 is found to be 71 feet, and thus with wheel 2 at the left end, wheel 14 is 3 feet = 74 − 71 from the right end. The moment of these loads about R_R is equal to the moment about wheel 14 plus the product of the sum of the loads by the additional lever arm. (This may be proved as on page 189 : 1.) The moment of wheels 2–14 about wheel 14 is found by following down the vertical line under wheel 14 to the heavy vertical line in the moment table, then toward the left to the number just to the right of the line through wheel 2, viz.: 11,900 thousand pound-feet. The sum of loads 2–14 is found similarly in the shear table to be 333 thousand pounds. The total moment is then 12,900 = 11,900 + 333 × 3 in thousands of pound-feet. The maximum shear is equal to the reaction R_L which is this total moment divided by the span, or 174 thousand pounds = 12,900 ÷ 74.

Illustrative Problem — Special Case. — To find the maximum shear on a 24-foot stringer for Cooper's E60 loading. If wheel 2 were placed at the left end, wheel 6 would be at R_R and only the four drivers would be on the span. (The distance 24 feet from wheel 2 to wheel 6 is not duplicated in the table because it is the same as from wheel 11 to wheel 15.) For beams between 23.0 and 27.3 feet long the maximum shear will equal the right reaction R_R when wheel 5 (or 14) is placed there, since not only the four drivers but the pilot wheel 1 (or 10) comes on the span. The parts of the shear and moment tables at the *right* of the zigzag line should be used in much the same way as the parts at the left. Thus, the distance from wheel 10 to wheel 14 is found to be 23, the sum of the loads 135, and the moment 1860. Hence, the maximum shear = R_R = 83 = (1860 + 135 × 1) ÷ 24 in thousands of pounds.

1. **Maximum Shear for Any Section** — *Cooper's Loading.* — The loading which will cause the maximum shear for any section of a simple beam depends upon the relative lengths of the segments. When the longer segment does not exceed 12.5 feet the special loading of two 37,500# wheel loads should be used. When the longer segment is between 23.0 and 27.3 feet and the shorter segment not over 9 feet, the maximum shear for the section will be found when wheels 1 to 5 are on the longer segment with wheel 5 next to the section, no load being on the shorter segment. For all beams over 34.5 feet long the maximum shear for any section will be found when the longer segment is fully loaded and wheel 2 is next to the section (wheel 1 being on the shorter segment if the latter is more than 8 feet long). See Fig. 194. For all other spans the maximum shear for any section will be found when the longer segment is fully loaded and when either wheel 2 or the first wheel of the special loading is next to the section, depending upon the relative lengths of the segments. The shear for any section is not necessarily equal to the reaction, for when wheel 1 is on the shorter segment it must be deducted.

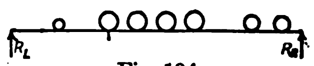


Fig. 194.

Illustrative Problem. — To find the maximum shear for a section 10 feet from the end of a 30-foot stringer for Cooper's E40 loading. The effects of two loadings must be compared because the beam is less than 34.5 feet long. Placing wheel 2 (or 11) next to the section, we find that wheel 5 (or 14) is 5 feet from the right end. The left reaction due to this position of wheels 1 to 5 is 64 = (1250 + 135 × 5) ÷ 30 for E60, and the corresponding shear is 49 thousand pounds = 64 − 15. Placing the first 37,500# wheel of the special loading next to the section, we find the shear, which is equal to the reaction, is only 41 thousand = 37,500 (13 + 20) ÷ 30. The maximum shear for E40 is therefore 33 = 49 × $\frac{2}{3}$.

2. **Maximum Floor-Beam Reaction** — *Cooper's Loading.* — In designing a through railway bridge it is necessary to determine the maximum concentrations on the floor beams. These are used in designing the floor beams and the girders and also the connections of the stringers to the

floor beams and the floor beams to the girders. The concentration on an intermediate floor beam at each stringer point is equal to the right-hand reaction of one stringer plus the left-hand reaction of the adjacent stringer. The loads should be placed to make this *sum* a maximum, but they cannot be placed to make both reactions maximum at the same time lest the engines overlap. The maximum concentration or " floor-beam reaction " will be found when one of the inner drivers of the second engine (wheel 12 or 13) is placed directly over the floor beam. It is usually necessary to try both wheel 12 and wheel 13 over the floor beam in order to ascertain which will give the larger floor-beam reaction. A criterion is sometimes used to determine which wheel to place over the floor beam. There is not much advantage to be gained by its use because both wheels 12 and 13 often satisfy the criterion and both must be tried the same as if no criterion were used. Care should be taken that the load over the floor beam is considered once, but only once, in finding the floor-beam reaction. It may be included in the right reaction of the left-hand span, or in the left reaction of the right-hand span, or it may be considered independently.

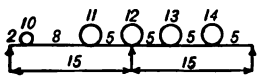


Fig. 195 (*a*).

Illustrative Problem. — To find the maximum floor-beam reaction at each stringer point in a bridge with 15-foot panels, center to center of floor beams, for Cooper's E60 loading. Placing wheel 12 over the floor beam, as in Fig. 195 (*a*); the reaction at this floor beam may be found from the table as follows:

$22.0 = (240 + 45 \times 2) \div 15 = R_R$
$30.0 = (150 + 60 \times 5) \div 15 = R_L$
$\underline{30.0}$ = load at floor beam
82.0 = total floor-beam reaction in thousands of pounds.

This result may be obtained without the table by combining the lever arms of equal loads for both spans, thus:

$$82.0 = [15.0 \times 2 + 30.0(10 + 15 + 10 + 5)] \div 15.$$

Similarly, by placing wheel 13 over the floor beam another value is obtained which in this case is smaller (81.3).

1. **Absolute Maximum Bending Moment** — *Cooper's Loading.* — The maximum bending moment for a simple beam not over 11.4 feet long will occur at the center when one of the two 37,500# wheel loads is placed at the center. For any simple beam or girder from 11.4 to about 90 feet long the absolute maximum bending moment will occur under one of the inner drivers of the second engine (wheel 12 or 13) when the distance between this driver and the center of gravity of all the loads on the beam is bisected by the center of the beam (page 191 : 1). The critical wheel will be adjacent to the center of gravity, usually the wheel nearest the center of gravity. The first engine will have an equal effect for spans up to about 50 feet, but never a greater effect. For girders longer than about 70 feet a portion of the uniformly distributed train load must be considered. The relative position of the center of gravity then changes with every movement of the loads, and more trials are usually needed to determine the proper position of the loads. For girders longer than about 90 feet the critical wheel should be determined from the criterion (page 191 : 1).

Illustrative Problem. — To find the maximum bending moment on a 40-foot girder for Cooper's E50 loading. By plotting the length to a scale of $\frac{1''}{16} = 1'$ ($40 \div 16 = 2\frac{1}{2}$ inches) on the edge of a strip of paper and sliding it along under the wheels in the table to include different combinations of wheels, it is found that the inner drivers 12 and 13 are brought in the vicinity of the center when wheels 10–16 are on the girder. The position of the center of gravity of these wheels may be found from the corresponding wheels of the first engine (wheels 1–7) to be 0.4 of a foot to the right of wheel 13.

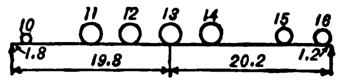


Fig. 195 (*b*).

The critical wheel is always adjacent to the center of gravity, and for spans less than 90 feet it is one of the inner drivers; in this case, therefore, the maximum bending moment will occur under wheel 13 when it is placed 0.2 foot = $0.4 \div 2$ to the left of the center of the span, as shown in Fig. 195 (*b*), wheel 16 being 1.2 feet from the right end. For E60 loading, the moment of all wheels about wheel 16 is 3230 thousand pound-feet, and the sum of the loads 10–16 is 174 thousand pounds. The left reaction is 86 thousand pounds $= (3230 + 174 \times 1.2) \div 40$. The bending moment under wheel 13 is 980 thousand pound-feet $= 86 \times 19.8 - 720$, the value 720 being the

moment of wheels 10–13 taken directly from the moment table. The bending moment for E50 loading is $\frac{5}{6}$ of 980 or 820 thousand pound-feet. In the use of the table, consistent accuracy should be used.

1. **Maximum Bending Moment at Any Point** — *Cooper's Loading.* — The maximum bending moment at any point of a beam or girder will occur when the critical wheel is placed at that point. For spans up to 100 feet the critical wheel will be either wheel 12 or wheel 13, although in some cases the maximum moment will occur when the engines face the longer segment instead of the shorter. For spans over 100 feet the critical wheel * should be found from the criterion on page 191 : 1.

2. **Through Girders** — *Cooper's Loading.* — The live loads in a through bridge are applied to each girder in the form of concentrated loads at the floor-beam connections. The action of the wheel loads may be best shown by sketching the stringers as if they rested on top of the floor beams, and the floor beams as if they rested on top of the girders, as in Fig. 196, although in reality web connections are used and the trains pass "through" the bridge between the girders. The end stringers may rest directly upon the abutments or they may be connected to end floor beams; in either case the effect upon the girder is virtually the same, except for the details at the ends. If the bridge supports a single straight track, each floor beam is symmetrically loaded and the corresponding girder load (for live loads) is numerically equal to the floor-beam reaction. The maximum floor-beam reaction cannot occur at more than one floor

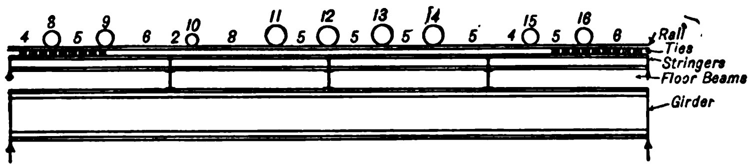


Fig. 196.

* The critical wheels for different segments are indicated in a table in Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

beam at the same time, and it is necessary to so place the loads that the bending moment at the floor beam nearest the center of the girder will be a maximum. It so happens that the bending moment at any panel point is the same as if the wheel loads were applied directly to the top flange of the girder, and hence, the position of the loads which will cause the maximum bending moment at any floor beam is found as in the preceding paragraph. The bending moment at this point may be found as in a deck girder or else from total concentrated loads obtained by combining the different floor-beam reactions with the corresponding dead loads (page 225 : 1). The maximum shear in the end panel will occur when the loads are placed to give a maximum bending moment at the first floor-beam point.

Illustrative Problem. — To find the maximum bending moment on a girder of a through railway bridge of four 15-foot panels for Cooper's E60 loading. The maximum bending moment will occur at the center floor beam when the critical wheel (either 12 or 13) is placed at that point. Let us assume that wheel 12 is the critical wheel † and that the loads are placed as in Fig. 196. The floor-beam reaction at the center floor beam is 82,000# as determined on page 194 : 2. In a similar manner the corresponding floor-beam reactions at the first and third quarter points are found to be 39,900# and 52,100# respectively. The bending moment for these loads combined with impact and dead loads is found on page 225 : 1, and the shear on page 250 : 3.

3. **Trusses** — *Cooper's Loading.* — Trusses may be designed for Cooper's loading or for equivalent uniformly distributed loads. The table on page 318 may be used to advantage in finding the live-load stresses in trusses, but the placement of the loads is beyond the scope of this book.‡

† Both positions should be tried to determine which causes a maximum. In this case a slightly greater bending moment is found when wheel 13 is placed at the center, but the difference is so small it is disregarded in order to simplify different steps in a series of problems based upon the values given here.

‡ See Marburg's "Framed Structures and Girders," Part I, McGraw-Hill Book Co., Inc., New York.

CHAPTER XXXI

THE DESIGN OF BEAMS

SYNOPSIS: Beams are proportioned according to the bending moment and shear determined from the external forces. The shape and the size of the cross section are designed with due regard to bending, shearing, buckling, bearing, and deflection.

1. **General.** — Most structures are designed in a Designing Department, although draftsmen are often called upon to design beams. Every draftsman should be familiar with the methods of design if for no other reason than to design the connections properly. The general arrangement of the beams in a structure is usually determined in the designing room. Beams which support machinery, tracks, runways, walls, etc., must be located to meet the given conditions. Beams near openings in floors or walls must be placed in the proper relation to these openings. The spacing of floor beams and roof beams or purlins in buildings depends upon the type of flooring or roofing and upon the superimposed loads. The type and the length of a beam, the form of loading, the magnitude of the loads, and the distance between beams all have their effect upon the shear and bending moment, as explained in the preceding chapter. After these are once determined the design of the beam, or the determination of the proper cross section, is the same for all, regardless of the sign of the shear or the bending moment.

2. **Points Considered.** — Beams should be designed to give proper resistance to bending, shearing, buckling, and deflection. Beams which rest upon other beams or walls must have sufficient bearing area (page 203 : 1). Beams are usually designed first to resist bending, and then the resistance of the resulting beam to vertical and horizontal shear is investigated. Beams seldom buckle except under a heavy concentrated load; this part of the design will be discussed under grillage beams (page 93 : 3). The amount of deflection is usually immaterial unless the beam is to support plastered ceilings, shafting, or machinery; when necessary, the deflection should be determined as on page 203 : 2.

3. **Effects of Bending.** — When a simple horizontal beam is loaded it sags, deflects, or bends downward. The horizontal fibers in the lower part of the beam are lengthened, while those in the upper part are shortened; between these two parts is a neutral surface in which the fiber lengths remain unchanged. Beams are designed upon the assumption that all points which lie in a transverse plane before a beam is bent remain in a plane after the beam is bent. In any fiber the "strain" or the change in length is proportional, within the elastic limit, to the "stress" or internal force which causes the strain, according to Hooke's law. The unit stresses used in the design of beams are well within the elastic limit (page 10) and this relation has been found by experiments to be true. It follows, therefore, that the horizontal fiber stresses are proportional to the distances of the fibers from the neutral surface. Stresses which tend to lengthen the fibers are called tensile stresses and the fibers are said to be in tension; similarly, compressive stresses tend to shorten the fibers which are in compression. When a beam is considered cut by an imaginary transverse section plane the external forces acting upon either segment are not in equilibrium by themselves but they are held in equilibrium by the internal forces acting in the fibers which are cut by the section plane. The intersection of the neutral surface by this section plane is termed the neutral axis of this cross section; this neutral axis passes through the center of gravity

For convenience let us consider a simple horizontal beam with vertical loads. This simplifies the phraseology and covers the great majority of beams; the principles can be readily adapted to other conditions. The internal force in each fiber cut by the section plane may be resolved into horizontal and vertical components, the latter all acting in the same direction. The sum of the vertical components must equal numerically the shear on the segment, because the algebraic sum of the vertical components of these internal forces and the vertical components of the external forces (i.e., the shear, page 184:1) must equal zero in order to satisfy the V equation of equilibrium. In like manner the H equation is satisfied when the algebraic sum of the horizontal components of both external and internal forces equals zero; since there are no horizontal components of external forces it follows that the sum of the horizontal compressive stresses above the neutral axis must equal numerically the sum of the horizontal tensile stresses below the neutral axis. In order to satisfy the M equation the resisting moment or the sum of the moments of the stresses in the fibers cut by the section plane must equal numerically the bending moment or the sum of the moments of the external forces acting on the segment (page 184 : 2). Since the point of moments is taken in the section plane (page 184 : 2), the lever arm of the vertical components of the internal forces is zero, so only the horizontal components need be considered. The significance of these points will now be discussed in detail.

1. **Resisting Moment — Theory.** — A portion of a beam cut by a vertical section is shown in Fig. 198, the right-hand portion of the beam being removed. The inclined line shows the relative position, greatly exaggerated, of the same section after the beam is bent. The arrows indicate the horizontal components of the stresses in the fibers cut by the section, being proportional to their distances from the neutral axis. The vertical components are not shown because they do not enter into the resisting moment (see preceding paragraph). The fiber which is farthest from the neutral axis (either in tension or compression) is stressed the most and consequently its strength determines the strength of the beam. The unit stress in this extreme fiber must not exceed the allowed unit stress for bending.

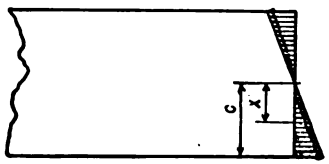


Fig. 198.

Let f = the allowed stress in the extreme fiber, in pounds per square inch,

a = the area of each fiber, in square inches,

c = the distance from the neutral axis to the extreme fiber, in inches, and

x = the distance from the neutral axis to any fiber, in inches.

Then fa = the stress in the extreme fiber,

$\frac{fax}{c}$ = the stress in any fiber at a distance x from the neutral axis,

$\frac{fax^2}{c}$ = the moment of this stress,

$\Sigma \frac{fax^2}{c} = \frac{f}{c} \Sigma ax^2$ = the sum of the moments of the stresses in all the fibers in the cross section, i.e., the resisting moment.

But the expression Σax^2 is generally recognized as the moment of inertia* of the cross section represented by I, therefore

$\frac{fI}{c} = m_R$ = the general expression for the resisting moment of any homogeneous beam.

2. In the above expression for resisting moment the **unit stress** f depends upon the material of which the beam is to be made. Values for f must be taken from the specifications which govern any specific design. For convenience, the values recommended by the American Railway Engineering Association for wood are given on page 320 and for steel on page 317. The values for wood are for a better grade of lumber than is often used and the values allowed by the building laws of many cities are correspondingly lower. The unit stresses for steel have been quite generally adopted, although higher values are allowed for higher grade steel used in large bridges. These values for f are intended only for beams which are properly stayed against lateral flexure (page 201 : 2).

* See Kirkham's "Structural Engineering," McGraw-Hill Book Co., Inc., New York, or almost any book on the Calculus or Mechanics. For a proof without the Calculus, see page 199 : 3.

1. **Section Modulus.** — The I and the c in the expression for resisting moment both depend upon the form of the cross section of the beam. Values of $\frac{I}{c}$ may be tabulated for different cross sections regardless of the unit stress. This quantity $\frac{I}{c}$ is called the "section modulus" and is often represented by the letter S. In this book the lower-case letter s will be used consistent with the adoption of lower-case letters for units which involve inches, and capitals for those which involve feet. Hence $m_R = fs$. The section moduli for different sections are found in the common handbooks of the steel manufacturers and also in the tables at the end of this book, as explained below.

2. **Units.** — An expression for resisting moment may be equated to an expression for bending moment provided they are in the same units, thus: $M_B = M_R$ or $m_B = m_R$ whence $m_B = \frac{fI}{c}$. Beams are usually of constant cross section and they are therefore designed for the maximum bending moment. If a beam is of variable cross section, such as an I-beam with plates riveted to the flanges along the central part of the beam, the resisting moment at any section must satisfy the maximum bending moment that can occur at that section. In the above equation, or "flexure formula" as it is sometimes called, there can be only one unknown quantity but that may be on either side of the equation. If the section modulus $\left(s = \frac{I}{c}\right)$ is unknown the problem is one in design, otherwise it is one in investigation. In design it is desired to find the shape and size of a beam which will meet given requirements. In investigation it is desired to find how great a load a given beam will support, how great a distance the beam will span, how far from the support a given load may be placed, or how great a unit stress is developed. The usual units for bending moments are pound-feet (page 184 : 2), while those for resisting moments are pound-inches (compare page 3 : 2). These must be made identical before they are equated, the value in pound-feet being multiplied by 12 to give pound-inches. Lack of care on this point accounts for the majority of all mistakes in the design of beams by beginners. A convenient method of determining the resulting unit of any expression such as that for resisting moment is to substitute the units of the component parts in the expression, and cancel like quantities which occur in both the numerator and denominator. This can be quickly done and it is less confusing than to combine the units and the numerical values in the same expression. For example, in the expression $\frac{fI}{c}$, f is in pounds per square inch, I is in inches to the fourth power, and c is in inches. The resulting unit is pound-inches, found as follows: $\frac{\#}{\text{in. in.}} \times \text{in. in. in. in.} \times \frac{1}{\text{in.}}$ or better $\frac{\#\text{in. in. in. in.}}{\text{in. in. in.}}$. Similarly, the unit of section modulus is inches cubed, so the unit of fs is pound-inches thus: $\frac{\#\text{in. in. in.}}{\text{in. in.}}$. This method can be applied in like manner to expressions for bending moment.

3. **Rectangular Beams.** — The moment of inertia of a rectangle is $\frac{bd^3}{12}$ where d is the dimension at right angles to the neutral axis and b the dimension parallel to this axis, both in inches. The distance c from the neutral axis to the extreme fiber is $\frac{d}{2}$ because the neutral axis through the center of gravity passes through the center of the rectangle. Substituting these values in the general expression, we find the resisting moment of a beam of rectangular cross section to be $\frac{fbd^2}{6}$, whence the section modulus is $\frac{bd^2}{6}$. This expression for resisting moment may be derived independently without the use of the Calculus. The forces above the neutral axis (Fig. 198) may be replaced by a single resultant force. The unit stress at the extreme fiber is f and at the neutral axis zero. Between these two the forces vary directly with the distance from the neutral axis (page 198:1). For unit breadth the sum of these forces is equivalent to the area of a triangle the base of which is f, and the altitude $\frac{d}{2}$. For a beam of breadth b the magnitude of this resultant compressive force is $\frac{1}{2} \times f \times \frac{d}{2} \times b$ which acts at the center of gravity of the triangle or

$\frac{1}{3} \times \frac{d}{2}$ below the top of the beam. Similarly, the forces below the neutral axis may be replaced by a single resultant tensile force equal in magnitude to the resultant compressive force and acting at a corresponding distance above the bottom of the beam. These two equal and opposite forces form a "couple," the moment of which about any point of moments is equal to the product of one force and the perpendicular distance between the two forces. This moment is the resisting moment of the beam and is equal to $\frac{1}{4}fbd \times \left(d - 2 \times \frac{d}{6}\right) = \frac{fbd^2}{6}$. This expression applies to homogeneous beams only; it does not apply to compound beams, such as reinforced concrete beams. Most of the homogeneous rectangular beams which are designed by the structural designer are wooden beams. These beams are quite often designed according to nominal dimensions, but it seems wiser and more logical to use the actual dimensions which in many cases are considerably less. This depends upon the custom followed in the saw mills of the different localities. By some the full nominal dimensions are furnished, while by others the width of each saw cut is deducted. If beams are planed, further deduction is made. The designer should be cognizant of the customs of the lumber mills which are most likely to furnish his material. In the absence of further information the following quite common grading rules may be used: rough sawed beams must be accepted if not more than $\frac{1}{4}''$ scant on each dimension, as $5\frac{3}{4} \times 11\frac{3}{4}$; surfacing or planing removes an additional $\frac{1}{8}''$ for each face surfaced, as $5\frac{5}{8} \times 11\frac{5}{8}$ if surfaced on one side and one edge (indicated S1S1E), or $5\frac{1}{2} \times 11\frac{1}{2}$ if surfaced on all four sides (indicated S4S). Tables of section moduli for both nominal and actual dimensions are given on page 319. In each case the beam is known by its nominal size and is so referred to. In general there will be several different sizes of beams which will furnish a required section modulus, unless one dimension is determined by fixed conditions, as for example, when a beam must be of the same depth as another similar beam. In case a table of section moduli is not available, a value of d (or b) must be assumed and the corresponding value of b (or d) calculated. Several trials may be made before a properly proportioned beam results. A knowledge of the stock sizes of the locality will assist in the selection of the beam to be used. The position in the structure, the method of support, and the tendency to overturn or to buckle transversely must all be considered. The deeper beams deflect less, are stiffer, and have a smaller cross section for the same strength, but they are more liable to overturn and they may not have sufficient bearing area to properly distribute the loads (page 203:1). In general the depth should be from $1\frac{1}{2}$ to 2 times the breadth.

1. For **cylindrical beams** of circular cross section $I = \frac{\pi d^4}{64}$, and $c = \frac{d}{2}$ where d is the diameter in inches. The resisting moment $m_R = \frac{f\pi d^3}{32} = 0.098fd^3$ and the section modulus $s = \frac{\pi d^3}{32} = 0.098d^3$.

2. **Steel Beams.** — In determining the moments of inertia of I-beams, channels, angles, and other structural shapes, the curves are neglected. It is unnecessary for the designer to use the cumbersome expressions * for the moments of inertia of these cross sections because numerical values for I and for s about axes perpendicular to each other are tabulated in the common handbooks and in this book (pages 322 to 326). I-beams and channels are regularly placed with their webs parallel to the applied forces. In the selection of a steel I-beam or channel to satisfy a given section modulus, preference should be given to the weights printed in larger type because these sections can usually be obtained more readily from the mills. Furthermore, it will happen frequently that these beams will weigh less per foot and therefore cost less than some other sizes of the same strength. Sometimes beams must be of the same depth as some other beams, and it is usually of advantage to reduce to a minimum the number of different sizes of beams for any one contract. The practical designer should keep posted upon market conditions and know which sizes are most easily and quickly obtained.

3. Every beam must be **self supporting,** and this fact must be considered in the design. Since the weight of a beam is not known until after the beam is designed, the weight must be assumed and the corresponding bending moment must be combined with the bending moment for the other dead and live loads. This is simpler than to attempt to carry the

* These expressions are given in the Handbook of the Cambria Steel Company.

weight of the beam in terms of the unknown dimensions. After the size of the beam is determined the actual weight should be compared with the assumed weight and the design should be corrected accordingly, if necessary. Usually the bending moment due to the weight of the beam forms a small portion of the total bending moment, so that a slight discrepancy does not matter. Whether or not the difference in weight is great enough to affect the design can usually be told by inspection. It does not take much experience to enable one to estimate the weight more closely than if neglected altogether and there is a good chance of one's guessing accurately enough to make a redesign unnecessary. In estimating the weight of a wooden beam it is usual to allow $3\frac{1}{2}$, 4, $4\frac{1}{2}$, or 5 pounds per foot *board measure*, the last two being for creosoted wood. One "foot board measure" (Ft.B.M.) is one-twelfth of a cubic foot, as in a board one foot square and one inch thick. To obtain the number of feet board measure in a piece of timber take two dimensions in feet and one in inches. Four pounds per foot is an average value for structural timber. If the dimensions of the cross section (b and d) are taken in inches the weight of a beam per linear foot is $4 \times 1 \times \frac{bd}{12}$ or $\frac{bd}{3}$. Upon this basis the weights in the last column on page 319 are obtained.

1. **A beam is weakened by holes.**—The size of the beam is determined by the maximum bending moment. Since a beam is generally of uniform cross section throughout its length there is an excess in area except near the point of maximum bending moment. Where this excess is large, as for example near the ends of a simple beam, holes may be safely punched or bored. When holes are required in wooden beams or in the flanges of steel beams where the bending moment is nearly maximum their effect should be considered. The moment of inertia of the net section should be used. This may be found by subtracting the moment of inertia of the holes from the moment of inertia of the cross section about the same neutral axis.

2. **Lateral Supports.**—Steel beams are designed for the usual unit stress of 16,000#/sq. in. in bending only upon the assumption that they are properly stayed laterally so that the compression flanges will not buckle. Such lateral support may be furnished by wooden flooring or sheathing, by any form of solid floor construction, by masonry walls, by tie rods, by special struts, by lattice bars or tie plates, by separators, diaphragms, etc. In the absence of some such support the allowed unit stress should be reduced according to the ratio of the distance (l) between lateral supports to the extreme width of member (b). The American Railway Engineering Association specifies that the maximum stress in the compression fibers must not exceed $16{,}000 - 200\frac{l}{b}$. For the sake of comparison, numerical values are tabulated below for this reduction formula, and also for the formulas recommended by R. Fleming * of the American Bridge Company, by the Carnegie Steel Company, and by the Cambria Steel Company. No value should exceed 16,000.

UNIT STRESSES FOR BEAMS WITHOUT LATERAL SUPPORT

Ratio $\frac{l}{b}$	A.R.E.A. $16{,}000 - 200\frac{l}{b}$	Fleming $19{,}000 - 250\frac{l}{b}$	Carnegie $19{,}000 - 300\frac{l}{b}$	Cambria $\dfrac{18{,}000}{1 + \frac{l^2}{3000b^2}}$
5	15,000	16,000	16,000	16,000
10	14,000	16,000	16,000	16,000
15	13,000	15,250	14,500	16,000
20	12,000	14,000	13,000	15,880
25	11,000	12,750	11,500	14,900
30	10,000	11,500	10,000	13,850
35	9,000	10,250	8,500	12,780
40	8,000	9,000	7,000	11,740

3. Beams which are subjected to a **lateral thrust** should receive special consideration. For example, a floor beam next to an elevator shaft or other opening may receive the thrust from a brick or tile floor arch on one side only. Intermediate beams have the thrust of one arch largely counterbalanced by that of the arch on the opposite side. The thrust of an arch at the ends of the arch span is equal to the maximum total compressive stress and this must be counteracted by the tension in tie rods or by the lateral resistance of the beams. Concrete floor slabs are usually reinforced so that no thrust on the beam need be considered. In deriving an expression for thrust in pounds per linear foot of beam

* *Engineering News*, April 6, 1916.

let us assume an equal tension T in the tie rods, also in pounds per linear foot of beam. These equal forces constitute a couple, and the perpendicular distance between them is the effective depth of the arch h in inches. The resisting moment Th must equal the bending moment on the arch due to a vertical load of U' pounds per *square* foot for a span of X feet center to center of beams, thus: $Th = 12\frac{U'}{2}\left(\frac{X}{2}\right)^2$ whence the thrust $T = \frac{3\,U'X^2}{2h}$. For flat arches commonly used, the effective depth may be taken 2.4″ less than the depth of the arch. A beam without lateral support must be designed so that the combined fiber stress due to vertical bending and to lateral bending will not exceed the allowed stress per square inch (16,000). A beam somewhat larger than the size required for vertical forces alone should be assumed and then investigated. The fiber stress due to vertical forces may be found by equating the bending moment of vertical forces to the resisting moment and solving for f, the section modulus of the assumed beam being known. Similarly, the fiber stress due to lateral forces may be found, the bending moment being found for the uniformly distributed thrust T (see above) and for the proper length between supports. The section modulus must be taken about an axis parallel to the web. Values for this s_2 for channels are given on page 323. Values for I-beams may be obtained from I_2 (page 322) and from c which is one-half the flange width. (Note that c for channels is the distance from the center of gravity to the further edge of the flange.) If a beam is supported by tie rods it becomes a continuous beam. By the Theorem of Three Moments an expression * may be found for the maximum lateral bending moment in terms of the thrust T and the panel length B between rods. If only one rod is used in the center of the span L, the bending moment is $\frac{TB^2}{8}$; if two rods are used making three equal panels of B feet the bending moment is $\frac{TB^2}{10}$; if three rods are used making four equal panels the bending moment is $\frac{3TB^2}{28}$.

* See Johnson-Bryan-Turneaure's "Modern Framed Structures," Part II, John Wiley and Sons, Inc., New York. See also footnote, page 193.

1. **Shear.** — After a beam is designed to give proper resistance to bending, its resistance to shear should be investigated. The size of the beam will seldom be increased as a result of this investigation, nevertheless it should be disregarded only by a man of experience when he is confident that no increase will result. The intensity of vertical shear at any point of a beam is equal to the intensity of horizontal shear at the same point,† much as the vertical and the horizontal water pressures at any point in a tank of water are equal. This intensity is not uniform throughout the cross section but is zero at the extreme fibers and maximum at the neutral axis. The maximum shear intensity in any beam should not exceed the allowed unit stress either in horizontal shear or in vertical shear. For steel the allowed stresses are equal, but for wood the value specified for the " longitudinal shear in beam " is considerably less than for the transverse shear. For any cross section of a rectangular beam the maximum shear intensity is three-halves of the average shear intensity, or $v = \frac{3V}{2bd}$, in which v = maximum horizontal or vertical shear intensity in pounds per square inch, V = maximum shear for the section in pounds, b and d = the breadth and depth of the beam in inches. This expression may be derived in the following manner. In Fig. 198 are shown the horizontal forces acting in the fibers of a beam cut by a transverse section. Let a similar section be passed at a small distance x from the first section.

Let f_1 = the unit stress in the extreme fiber at the first section,
f_2 = " " " " " " " " " second "
$c = \frac{d}{2}$ = the distance from the neutral axis to the extreme fiber.
$m_1 = f_1 s$ = the bending moment at the first section,
$m_2 = f_2 s$ = " " " " " second "
$m_2 = m_1 + Vx$ (page 189 : 1), the shear virtually being constant between sections,
$s = \frac{bd^2}{6}$ (page 199 : 3).
$f_2 - f_1$ = the increase in stress in the extreme fiber within the distance x.

† See Kirkham's "Structural Engineering," McGraw-Hill Book Co., Inc., New York, Fuller and Johnston's "Applied Mechanics," Vol. II, John Wiley and Sons, Inc., New York, or similar books.

There is a proportionate increase in every fiber between the extreme fiber and the neutral axis. The longitudinal shearing stress at the neutral axis for the distance x per unit of breadth is the sum of the increases in stress in all these fibers. This sum is equivalent to the area of a triangle of which the base is $f_2 - f_1$ and the altitude $\frac{d}{2}$. The shearing stress per square inch or the shear intensity is equal to this sum divided by x, or

$$v = \frac{\frac{1}{2}(f_2 - f_1)\frac{d}{2}}{x} = \frac{\frac{1}{2}(m_1 + Vx - m_1)\frac{d}{2}}{\frac{1}{6}bd^2x} = \frac{3V}{2bd}.$$

The general expression* for the shear intensity at any point of a beam of any cross section is $v = \frac{Vq}{Ib}$, in which q = the statical moment about the neutral axis of that portion of the cross section between a horizontal plane through the extreme fiber and a horizontal plane through the given point, and b = the breadth of the beam where cut by the latter plane. The statical moment is the product of the area of the portion of the cross section referred to by the distance from its center of gravity to the neutral axis. Thus the expression for the maximum shear intensity at the center of a rectangular beam may be found from the general expression as follows: $v = \frac{V \times \frac{bd}{2} \times \frac{d}{4}}{\frac{bd^3}{12} \times b} = \frac{3V}{2bd}$. For beams of circular cross section the maximum shear intensity is four-thirds of the average intensity or $v = \frac{16V}{3\pi d^2} = \frac{4V}{3\pi r^2}$ in which d and r = the diameter and the radius of the beam in inches. For I-beams and channels there is no convenient expression for the maximum shear intensity, nor can the intensity be found without considerable computation.† For all practical purposes, however, it is sufficiently accurate to use an approximate method which can be applied much more easily. The web must furnish most of the resistance to shear because the flange areas are concentrated near the extreme fibers where the shear intensity is small. The maximum shear intensity for an I-beam or a channel may be found by dividing the maximum shear V by the area of the straight portion of the web between the flanges. This area is equal to the web thickness multiplied by the tangent distance ($d-2k$ from the tables) between the curved fillets.

1. **Bearing.** — A beam which rests upon its supports must have sufficient bearing area to properly distribute the pressure of the beam on the supporting wall, beam, or column. A beam which rests upon masonry walls usually has a steel or cast-iron bearing plate to provide greater bearing area. The design of such a bearing plate is explained in Chapter XLIII, page 288. The bearing is not a determining factor in other methods of support for steel beams, such as connection angles with rivets. The bearing does play an important part in the design of wooden beams because it may determine the width of the supported beam or the width of the beam or column which supports it. The bearing area or the horizontal area of contact between a beam and its support must be large enough to prevent the fibers from crushing. The allowed pressure per square inch is the unit stress in "crushing" or compression at right angles to the grain. The total pressure is equal to the maximum reaction which can be caused by the full dead load (including the weight of the beam) and the live load. The required bearing area is found by dividing this total pressure by the unit stress in crushing *across* the grain. The necessary length of bearing or the distance which the beam must project onto the support is found by dividing the bearing area by the width of the supported beam. If two beams are supported at the same point of a supporting beam, the width of the latter must be sufficient to furnish the proper length of bearing for both beams. Sometimes two narrow joists or beams overlap so that each may bear on more than one half the width of the supporting beam.

2. **Deflection.** — The amount of vertical deflection is an important consideration only in the design of beams which support tile or concrete floors, plastered ceilings, shafting, etc. If the deflection exceeds a certain amount, unsightly cracks are liable to result in the floors or ceilings, and free action of the shafting or other moving parts may be impaired. Certain limiting ratios of the depth to the length of a beam can be found,

* See Fuller and Johnston's "Applied Mechanics," Vol. II, John Wiley and Sons, Inc., New York.

† See Fuller and Johnston's "Applied Mechanics," Vol. II, John Wiley and Sons, Inc., New York, or the Carnegie Steel Company's "Pocket Companion."

above which a beam will not have excessive deflection. By means of these ratios the designer can properly design beams without further investigation for deflection. Accordingly, the theory of the elastic curve will not be developed here,* but sufficient formulas will be given to enable the student to determine the amount of deflection in the average beam. In all of these expressions, L = the effective length of the beam in feet, l = the same in inches, P = the concentrated load in pounds, U = the unit load uniformly distributed in pounds per linear foot, W = the corresponding total load uniformly distributed in pounds, I = the moment of inertia of the cross section in inches^4, and E = the modulus of elasticity in pounds per square inch. The modulus of elasticity for steel is 29,000,000; values for different kinds of wood are given on page 320. The maximum deflection in inches at the center of a simple beam under a full load uniformly distributed is $\frac{5Wl^3}{384EI} = \frac{45UL^4}{2EI}$, and under a single load concentrated at the center is $\frac{Pl^3}{48EI} = \frac{36PL^3}{EI}$. The maximum deflection in inches at the free end of a cantilever beam supported at one end only under a full load uniformly distributed is $\frac{Wl^3}{8EI} = \frac{216\,UL^4}{EI}$, and under a single load concentrated at the free end is $\frac{Pl^3}{3EI} = \frac{576PL^3}{EI}$. For beams with combined loads the deflection due to concentrated loads and uniformly distributed loads may be added. It has been determined experimentally that plaster will crack when the deflection is more than $\frac{1}{360}$ of the span, and this value appears in many specifications. If we equate $\frac{12L}{360}$ to $\frac{45\,UL^4}{2EI}$ or to $\frac{36PL^3}{EI}$ and eliminate U or P as explained below, we can solve for the corresponding ratio of the depth to the length. By equating $m_B = \frac{12\,UL^2}{8}$ or $\frac{12PL}{4}$ to $m_R = \frac{fI}{c} = \frac{2fI}{d}$ we find $U = \frac{4fI}{3dL^2}$ and $P = \frac{2fI}{3dL}$. By substituting these values above and also E = 29,000,000 and f = 16,000, we can solve for $\frac{d}{12L} = \frac{d}{l}$. In this manner we find that for simple steel I-beams or channels designed for a unit stress of 16,000#/sq. in. in bending, and a modulus of elasticity of 29,000,000#/sq. in., the deflection will not exceed $\frac{1}{360}$ of the span if the depth is at least $\frac{1}{24}$ of the span for a uniformly distributed load or at least $\frac{1}{30}$ of the span for a single concentrated load at the center. In other words, the length in feet should not be more than two (or two and one-half) times the depth in inches. Other ratios can be derived in a similar manner as required. When a beam is subject to shocks or vibrations the depth should not be less than $\frac{1}{15}$ the span. I-beam stringers for railway bridges should preferably have a depth not less than $\frac{1}{12}$ of the span. When the depth cannot fulfill the above conditions the beam should be made enough stronger so that the deflection will not be greater than if a beam of the required depth were used. Roof purlins may usually have a depth of only $\frac{1}{30}$ of the span because the maximum load is seldom, if ever, realized.

1. The principal points of this chapter are illustrated by the following **typical problems.**

First Problem — Wooden Beam. — Design a 12-foot Norway Pine beam for a building, to resist a bending moment of 12,500#ft. and a shear of 4,200#.

12,500#ft. = bending moment of superimposed loads
400#ft. = $\frac{24}{2} \times 6^2$ = bending moment due to weight of beam
(6 × 12 assumed; see page 319 for weight)
12 × 12,900 = 1,200s (for unit stress in bending, see page 320)
129 = s.

Counting the actual dimensions $\frac{1}{4}''$ scant, this calls for a 6 × 12, the section modulus for a $5\frac{3}{4} \times 11\frac{3}{4}$ being 132.

4,200# = shear due to superimposed loads
150# = 24 × 6 = shear due to weight of beam

$$90\#/\text{sq. in.} = \frac{3 \times 4{,}350}{2 \times 5.75 \times 11.75} = \text{maximum shear intensity (page 202:1).}$$

This is safely under the 150 allowed for longitudinal shear in beams (page 320).

$$3.3'' = \frac{4{,}350}{230 \times 5.75} = \text{the length of bearing required.}$$

* See Fuller and Johnston's "Applied Mechanics," Vol. II, John Wiley and Sons, Inc., New York; or Kirkham's "Structural Engineering," McGraw-Hill Book Co., Inc., New York. For diagrams by C. A. Ellis, see *Engineering Record*, Jan. 15, 1916.

Second Problem — Wooden Beam. — Design a Long Leaf Yellow Pine (abbreviated LLYP) Highway bridge beam 20 feet long to support a uniformly distributed load of 250#ft.

$$30\#/\text{ft.} = \frac{8 \times 12}{3} = \text{weight of beam } 8 \times 12 \text{ assumed (page 200 : 3)}$$

$$14{,}000\#\text{ft.} = \frac{30 + 250}{2} \times 10^2 = \text{total } M_B$$

$$12 \times 14{,}000 = 1{,}630s$$

$$104 = s \text{ which calls for a } 6 \times 12 \text{ or an } 8 \times 10 \text{ although both are large.}$$

Since a 6×10 is nearly large enough it should be tried because the reduced weight may reduce the section modulus sufficiently.

$$20 = \frac{6 \times 10}{3} = \text{revised weight}$$

$$4 = \frac{(30 - 20)10^2 \times 12}{2 \times 1630} = \text{the reduction in } s.$$

$100 = 104 - 4 =$ the revised section modulas. A 6×10 can therefore be used.

$$2{,}700\# = (20 + 250)10 = \text{maximum shear.}$$

$$68\#/\text{sq. in.} = \frac{3 \times 2{,}700}{2 \times 6 \times 10} = \text{maximum shear intensity which is safely under the 150 allowed.}$$

$$1.2'' = \frac{45 \times 270 \times 20^4}{2 \times 1{,}610{,}000 \times 500} = \frac{45\,UL^4}{2EI} = \text{the maximum deflection.}$$

Third Problem — Steel Beam. — Design a steel beam to satisfy the conditions of the first problem above.

$$12{,}500\#\text{ft.} = \text{bending moment of superimposed loads}$$

$$300\#\text{ft.} = \tfrac{15}{2} \times 6^2 = M_B \text{ of assumed weight of beam}$$

$$12 \times 12{,}800 = 16{,}000s \text{ (for unit stress in bending, see page 317)}$$

$$9.6 = s.$$

Either a 7″ I 15# (page 322) or a 9″ $\sqcup$ 13¼# (page 323) can be used. Note that an 8″ $\sqcup$ 16¼# is strong enough, but it weighs more and is not so readily obtained (page 200 : 2).

$$4{,}200\# = \text{shear due to superimposed loads}$$

$$90\# = 15 \times 6 = \text{shear due to weight of } 7'' \text{ I } 15\#$$

$$3{,}270 = \frac{4{,}290}{.25(7 - 2 \times \frac{7}{8})} = \text{shear intensity which is safely under the}$$

10,000#/sq. in. allowed for beam webs as well as for girder webs (page 317).

Fourth Problem — Investigation. — Find the safe load concentrated at the center which a 15″ I 42# 30′–0″ can support. Also find the maximum deflection. The section modulus is 58.9 in.³ and the unit stress is 16,000#/sq. in. The reaction for a load P at the center is $\frac{P}{2}$ and the bending moment $\frac{P}{2} \times 15$. The bending moment due to the weight of the beam is $\frac{42}{2} \times 15^2$. Hence

$$12\left(\frac{P}{2} \times 15 + \frac{42}{2} \times 15^2\right) = 16{,}000 \times 58.9$$

$$9{,}840\# = P.$$

$$.75'' = \frac{36 \times 9{,}840 \times 30^3}{29{,}000{,}000 \times 442} = \frac{36PL^3}{EI} = \text{deflection due to concentrated load.}$$

$$.06'' = \frac{45 \times 42 \times 30^4}{2 \times 29{,}000{,}000 \times 442} = \frac{45\,UL^4}{2EI} = \text{deflection due to weight of beam.}$$

$$\overline{.81''} = \text{total deflection.}$$

CHAPTER XXXII

THE DESIGN OF TENSION AND COMPRESSION MEMBERS

Synopsis: The principles of design are given for the more common types of tension and compression members and for lattice bars. No attempt has been made to cover the practical points which must be considered in the design of a complete structure.

1. The structural designer must take into consideration many **practical points.** He must not design each member independently of other members, but he must so proportion all the members that they will form the best complete structure. He must consider not only the strength and the appearance of each member, but he must also anticipate the details so that the member can be fabricated and connected to other members to the best advantage. The design of complete structures is outside the scope of this book,* but the fundamentals of design are here given inasmuch as the draftsman is often called upon to design simple members.

2. **A tension member** is designed to transmit tensile stresses in a direction parallel to its principal axis; the stresses which are transmitted by a tension member tend to elongate that member. **A compression member** is designed to transmit compressive stresses in a direction parallel to its principal axis; the stresses which are transmitted by a compression member tend to shorten that member. Sometimes forces are applied to either tension or compression members which tend to bend them transversely. Such members must be designed for stresses due to both bending and direct tension or compression.

* For more complete treatises see Johnson-Bryan-Turneaure's "Modern Framed Structures," Vol. III, John Wiley and Sons, Inc., New York, for bridges and roof trusses; Ketchum's "Mill Buildings," McGraw-Hill Book Co., Inc., New York, for mill building construction; Burt's "Steel Construction," American Technical Society, Chicago, for office building construction; Kirkham's "Structural Engineering," McGraw-Hill Book Co., Inc., for bridges, mill buildings, and office buildings; and Kunz's "Design of Steel Bridges," McGraw-Hill Book Co., Inc., for viaducts, movable bridges, arches, cantilevers, etc.

3. **The area of cross section** is an important factor in the design of either a tension or a compression member, while the **form of cross section** is of special importance in a compression member. A tension member is equally strong whether the cross section is in the compact form of a circle or rectangle, or in the more open form of a hollow pipe or similar section, provided the area is the same. A compression member, on the other hand, is stronger if the metal is distributed so that the member is less likely to buckle or bend under compression. Thus a small rod would not resist so much compression as would a hollow pipe containing the same amount of metal. Cast iron columns, for example, are made hollow for this reason.

4. **Effect of Rivet Holes.** — A tension member is weakened by having holes punched in it for rivets. A compression member is not weakened in the same manner, provided the rivet holes are completely filled with either shop or field rivets. The reason for this difference is that the rivets are in *contact* with the metal surrounding the holes, but they are not *attached* to this metal. The rivets in the holes can therefore transmit compressive stresses from one side of the hole to the other much as the original metal would, but the rivets cannot prevent the member from pulling away from them when the member is subjected to tensile stresses. Rivets are compressed in driving, so the effect of cooling is to reduce the resulting pressure in the rivets rather than to shrink them so they are no longer in contact. Bolts, however, do not completely fill the holes. Any bolt holes, or holes left open, should be considered in determining the effective area of a compression member, provided they occur where

there is considerable tendency to buckle. In a tension member it is assumed that the total stress is distributed uniformly over the *net area;* in a compression member it is assumed that the total stress is distributed uniformly over the whole gross area, except as provided in the preceding sentence. These assumed conditions are not always fulfilled, particularly when the end connections are not properly designed; but the assumptions have proved sufficiently accurate for the design of such members as are discussed in this chapter.

TENSION MEMBERS

1. A tension member must be so designed that it is strong enough at its **weakest point** to carry the total stress. Some of the lightest tension members are made of round or square rods. Different methods are employed for fastening these rods to other members, as illustrated on page 316.

2. **A loop rod** is made by bending the end of a rod back upon the rod and forging it to form a loop (page 316). The loop is shaped to fit around a pin such as a cotter pin (Fig. 279) or a larger pin (Fig. 278 (*b*)). The loops are made stronger than the main part of the rod, so that the designer simply has to determine the size of the rod. Both round and square rods are used for loop rods. The required area of cross section is found by dividing the total stress by the allowed unit stress. From the table on page 315 the diameter of a commercial size of round rod may be selected with an area (second column) which equals or exceeds the required area. The size of a square rod may be found by taking the square root of the area or by using tables of square rods found in the handbooks of the different steel manufacturers. The sizes most used are multiples of $\frac{1}{8}''$.

3. **A clevis** is a forging made with two loops between which a connecting plate is inserted and held in place by a cotter pin. The clevis is made to screw on the end of the rod like a nut, a right-hand thread being used at one end and a left-hand thread at the other so that when the rod is turned it is tightened. Clevises are used less frequently than formerly because of the relatively high cost.

4. The form of rod most used is the **round rod threaded** at the ends for nuts, as shown in the different types of connection on page 316. The effective area of the rod is reduced when a thread is cut and a correspondingly larger rod must be used so that the least area at the root of the thread will be sufficient. In most cases a larger rod is used for the entire length. The ends of some of the longer rods are **upset** in the forge shop to a larger diameter so that after the threads are cut the strength at the ends is slightly greater than the strength of the main portion of the rods. The diameters and lengths of standard upset ends are given on page 315. The design of an upset rod is the same as for a loop rod, only the main portion of the rod being considered. The table of root areas for the upset ends may be used in the design of rods which are threaded without being upset. For example, let us design a rod to carry a stress of 7,500# at a unit stress of 16,000#/sq. in. The net area must be at least 0.47 sq. in. = 7500 ÷ 16,000. The diameter of a rod with a root area of 0.55 is 1″, which must be used as the next smaller rod has a root area of only 0.42. Note that a loop rod or an upset rod used under these conditions need be only $\frac{7}{8}''$ in diameter for the gross area is 0.60. The asterisks indicate that rods less than 1″ are not as a rule upset. The asterisks have no significance when the table is used for threaded rods not upset. In order to obtain the root area for any multiple of $\frac{1}{8}''$ between $\frac{3}{4}$ and 3 it becomes necessary to use the following values to supplement those in the table:

Diam. of Upset	Area as Root of Thread
$1\frac{1}{8}$	0.69
$1\frac{7}{8}$	2.05
$2\frac{3}{4}$	4.62

5. **Eye bars** are used for the tension members of pin-connected bridges. The ends are upset and punched, as shown in Fig. 40 (*d*), and then the holes are accurately bored to the proper size and at the proper distance apart. The heads are made the same thickness as the rest of the bar, and they are designed to fully develop the main body of the bar so that no bar tested to destruction should fail in the head. The design of an eye bar is therefore quite simple. Eye bars are usually arranged in pairs to keep the forces on the pins symmetrical. The depth or width of an eye bar is usually determined by the size of other eye bars and of other members

in the bridge. The thickness of each bar and the number of bars are determined by the required area of cross section, which is found by dividing the total stress by the unit stress. The thickness of each bar is usually between 1″ and 2″; 2″ is the maximum thickness, while the minimum thickness differs with different widths of bar and with different sizes of pins as indicated in the handbooks of the steel manufacturers. Between these limits the thicknesses vary by one-sixteenth of an inch.

1. **Riveted Tension Members.** — Tension members other than rods and eye bars are usually made of angles, channels, plates and angles, or plates and channels. One or two angles are used for the lighter members, as for example, those in latticed girders (Fig. 110), roof trusses (Fig. 116, or bracing (Fig. 140). Four angles are often used, as in the diagonal of Fig. 143. The component parts of a member must be fastened together to distribute the stress so that each part receives its share. Two angles are fastened at intervals by stitch rivets (page 69:4). Four angles may be fastened by batten plates, by lattice bars, or by continuous plates (Fig. 125, 122, or 126); continuous plates may be counted as part of the effective cross section.

2. The strength of a tension member is proportioned to the least **net area of cross section,** as explained on page 206:4. Riveted tension members are usually connected to other members by means of connection plates, and the least net areas are most often found through the holes for the rivets which connect the members to these plates. The weakest section of an ordinary light tension member is a cross section through the largest number of rivet holes. Compare page 209:1. The net area of this section is found by combining the net areas of the component parts. The net area of each part is found by subtracting the area of cross section of the holes from the gross area found from the tables. The area of cross section of a hole is the area of a *rectangle*, not of a circle (Fig. 221); it is the product of the diameter of the hole and the thickness of metal. The diameter of the hole in designing is taken $\frac{1}{8}''$ greater than the nominal diameter of the rivet. The hole is actually punched only $\frac{1}{16}''$ larger, but the metal around the hole is damaged during the process of punching so that it cannot be counted upon to carry the full stress per square inch; the practically universal method of taking this into account is to deduct an additional $\frac{1}{16}''$, calling the hole $\frac{1}{8}''$ larger than the rivet. The areas of holes are tabulated on page 303; these areas may be used in finding the net areas of angles, channels, or other shapes. The net areas and the strengths of tension members composed of two angles are given on page 327; the net areas and strengths of single angles may be found by dividing these values by two. The net area of a *plate* can be found more conveniently by multiplying the *net width* of the plate by the thickness, since both the area of the plate and the area of each hole are proportional to the thickness. The table on page 321 is arranged to give both net and gross areas of plates. The net area is found opposite the net width. Thus the net area of a 14″ plate with two holes deducted for $\frac{3}{4}''$ rivets is the same as the gross area of a $12\frac{1}{4}''$ plate.

3. **Illustrative Problem.** — *Investigation.* — Let us find the safe load of a section composed of 1 Pl. $12 \times \frac{3}{8}$ and 4 **L***s* $4 \times 3 \times \frac{3}{8}$ with holes for $\frac{3}{4}''$ rivets, arranged as in *C* 3, Fig. 137. Unit stress in tension = 16,000#/sq. in.

$$\begin{aligned}
8.60 \text{ sq. in.} &= 4(2.48 - \tfrac{7}{8} \times \tfrac{3}{8}) = \text{the net area of } 4\,\mathbf{L}s\,4 \times 3 \times \tfrac{3}{8}\\
\underline{3.84} \text{ sq. in.} &= (12 - 2 \times \tfrac{7}{8})\tfrac{3}{8} = \text{the net area of 1 Pl. } 12 \times \tfrac{3}{8}\\
12.44 \text{ sq. in.} &= \text{total net area}\\
199{,}000\# &= 12.44 \times 16{,}000 = \text{the safe load.}
\end{aligned}$$

The net areas should be indicated completely as shown for the sake of future reference; this is of special benefit in student work because the instructor can mark mistakes in such a manner that the student can tell whether he took the wrong number of holes, the wrong diameter, or the wrong thickness, or whether he took the wrong value from the tables or made an arithmetical mistake. Thus, in the first line are shown the number of angles, the gross area of one angle, the number of holes deducted from each angle (in this case 1), the diameter of the hole ($\frac{1}{8}''$ larger than the diameter of the rivet), the thickness of the hole (should be the same as the thickness of the angles), and the description of what the result indicates, i.e., the net area of a given number of angles of a certain size. The gross area is taken from the table on page 303 as is also the area of one hole, $0.33 = \frac{7}{8} \times \frac{3}{8}$. Similarly, in the second line are shown the full width of the plate, the number of holes, the diameter of each hole, the thickness of the plate, and the description of the result.

4. **The design** of a riveted tension member is an indirect process because the area of the holes cannot be found until the thickness of the

metal is known. After the required net area is found, the size of the section may be approximated and the corresponding actual net area may be determined; unless this actual net area equals or exceeds the required net area another trial should be made. The final section should be the *smallest* section which will satisfy the requirements. The following **typical problems** illustrate the design of simple tension members.

First Problem. — Design a single angle, with a row of $\frac{7}{8}''$ rivets in one leg, to support a load of 30,000# at a unit stress of 15,000#/sq. in.

2.00 sq. in. = 30,000 ÷ 15,000 = the net area required.

The simplest method is to use the table of net areas for two angles (page 327) for a value equal to *twice* the required area. With one hole for a $\frac{7}{8}''$ rivet deducted, the following sections will satisfy the requirements: $4 \times 4 \times \frac{5}{16}$, $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$, $3 \times 3 \times \frac{7}{16}$, $5 \times 3 \times \frac{5}{16}$, $4 \times 3 \times \frac{3}{8}$, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{7}{16}$. If the unit stress were 16,000 the problem would be still further simplified, since the stresses could be taken directly from the table and it would be unnecessary to find the net area.

Second Problem. — A hanger is composed of two 8″ channels placed back to back with a space between them for the insertion of connection plates at the ends. They are riveted to the plates by $\frac{3}{4}''$ rivets placed in three rows. The total stress is 100,000#, and the unit stress is 16,000#/sq. in. Design the member.

6.25 sq. in. = 100,000 ÷ 16,000 = the net area required.

The gross areas and web thicknesses of channels are found on page 323. Since the web thickness is not a multiple of $\frac{1}{16}''$ it is better to use the decimal; with a slide rule this is as convenient as to use the table of areas for rivet holes.

(5.54 sq. in. = 2(3.35 − 3 × $\frac{7}{8}$ × .22) = the net area of 2–8″⊔ 11¼#)
6.46 sq. in. = 2(4.04 − 3 × $\frac{7}{8}$ × .31) = " " " " 2–8″⊔ 13¾#

The result of the first trial was too small, so the second trial was necessary. Two 8″ ⊔ 13¾# would be used. It is a good plan to draw parentheses around all trial designs except the one adopted.

Third Problem. — Design a 12″ splice plate, with four lines of $\frac{7}{8}''$ rivets, to carry a stress of 65,000# at 16,000#/sq. in.

4.06 sq. in. = 65,000 ÷ 16,000 = the net area required
8.50 in. = 12 − 4 × $\frac{7}{8}$ = the net width of the plate
$\frac{1}{2}''$ = .48 = 4.06 ÷ 8.50 = the thickness required.

Note that the thicknesses of commercial plates vary by sixteenths, and unless the resulting thickness is a multiple of $\frac{1}{16}''$, the next higher value should be used. The length of the splice plate depends upon the total number of rivets required (page 270 : 2).

1. **The least net section is not necessarily a right section.** Rivets in another line may be spaced so close that a member tested to destruction would fail along a zigzag line, as shown in Fig. 209. The relative strength cannot be judged by comparing the full net section along the zig-zag line to the net right section, because the unit stress along the inclined lines is not the same as along the transverse lines. The maximum diagonal tension may be computed from the normal and the tangential components of the longitudinal stress. The minimum stagger which can be used without making the strength of the member less than at the net right section is found when this maximum diagonal unit stress equals the unit stress on the net right section. This minimum stagger may be found from the diagram on page 305 for different diameters of rivets and for different distances between rivet lines. The diagram is plotted for the following equation:

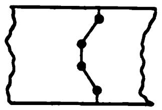


Fig. 209.

$$\left(\frac{g}{2} - \frac{d + \frac{1}{8}}{4}\right)\left(g + \sqrt{g^2 + 4f^2}\right) = g^2 + f^2 - \left(d + \tfrac{1}{8}\right)\sqrt{g^2 + f^2}$$

in which g = the gage or the transverse spacing from center to center of rivets (see the figure under the diagram), f = the minimum longitudinal pitch or stagger from center to center of rivets, and d = the nominal diameter of the rivets. This equation is based upon theory * which has been substantiated by tests on riveted tension splices.† If practicable, the pitch should not be less than the proper value found from the diagram. Otherwise, *a corresponding reduction in the net section must be made,* as explained in the next paragraph. Unfortunately, the importance

* Adapted from a similar expression derived by V. H. Cochrane in the *Engineering News,* April 23, 1908. See also the *Engineering News,* May 6, 1915.
† "Versuche im Eisenbau," Berlin, 1915.

of this requirement is not yet fully realized by all designers and writers of specifications.

1. **Working Rule for Effective Net Section.** — Obviously, holes that are staggered do not weaken a member as much as if the holes were all in the same right section. If a hole is as far from a given right section as the minimum stagger determined from the diagram (see preceding paragraph) it has no effect upon the strength; if the hole is placed in the right section the net section is reduced by the full area of the hole; if the hole is between these limits the net section is reduced by a fraction of the area of the hole. From the theory upon which the formula of the preceding paragraph is based may be found this fractional part of a rivet hole which should be deducted in order to give the actual effective net section.* This method is too cumbersome for general use, but approximately the same results may be obtained by means of a practical working rule † which is recommended; all variations are on the side of safety. This rule is as follows: "*The net section of a plate or shape shall be defined as the least section obtainable across the rivet holes, square or zigzag, taking every net distance in a diagonal direction at 85% of the value, except where 85% of the distance is less than the square projection, in which case the latter shall be used instead.*" The application of this rule may be illustrated by a $12 \times \frac{1}{2}$ plate with holes for $\frac{7}{8}''$ rivets, as shown in Fig. 209; the transverse spacing is 3″ with $1\frac{1}{2}''$ edge distances and the longitudinal spacing or stagger is 2″. The net area of a right section is 5.00 sq. in. $= (12 - 2 \times 1)\frac{1}{2}$; if the rivets were placed in the same row the net area would become $4.00 = (12 - 4 \times 1)\frac{1}{2}$. Since the stagger is less than the minimum of $2\frac{5}{8}$ found from the diagram on page 305 (for a gage of 3″), the effective net area must fall between 4.00 and 5.00 sq. in. The net area outside of the outer holes is $1.00 = 2(1\frac{1}{2} - \frac{1}{2})\frac{1}{2}$ and the net area between the inner holes is $1.00 = (3 - 1)\frac{1}{2}$. The diagonal distance from center to center of holes is 3.60; this may be determined graphically from a full-sized layout, from the diagram on page 312, or from a table of squares. Eighty-five per cent of the corresponding net area is $1.11 = 0.85(3.60 - 1.00)\frac{1}{2}$; this is not less than the "square projection" $1.00 = (3 - 2)\frac{1}{2}$, so two such areas should be combined with the net areas already found to make the total effective net area 4.22 sq. in. $= 2 \times 1.11 + 1.00 + 1.00$. It should be noted that the rivet spacing is a prerequisite to the application of this rule, and the designer must anticipate the details. He must assume the number of rivets in a right section and also the minimum stagger; these are sometimes apparent, but usually he must either specify them to the draftsman or else indicate the least net area for which a member is designed so that the draftsman can space the rivets accordingly. The design of some of the smaller members may be simplified by considering all holes in the same right section, unless they can be spaced at the minimum stagger determined from the diagram; if only one or two additional holes are involved, particularly if the stagger is small, very little metal is thus wasted.

* Derived by T. A. Smith in the *Engineering News*, May 6, 1915.

† Developed by D. B. Steinman, in the *Engineering News-Record*, June 14, 1917.

2. In the preceding paragraphs, the least net section is considered to be found within the portion of a member which is subjected to the full stress. **A smaller net section** may be safely used **near the ends** of a member where part of the stress has been transmitted to the connection plates by means of rivets. In this way the size of the connection plates may be reduced. For illustration, let us consider the strength of the member *L0–2* Fig. 125. The least net section designed to take the full stress is 30.12 sq. in. $= 2(21 - 4 \times 1)\frac{9}{16} + 4(3.25 - 1 \times \frac{1}{2})$ found through the four shop rivets (in each web) near the right end where the $14 \times \frac{9}{16}$ fillers begin. In the connection at the left end of the member, the net area through the three field rivet holes at the right of the group must be sufficient to take the full stress also. A smaller net area may be used two spaces to the left of this section, because the stress has been reduced by the strength of ten field rivets in single shear or 60,100# (page 310). Two rivets in the bottom batten plate come within $1\frac{1}{2}''$ of this new section; the minimum stagger from the diagram is $2\frac{3}{4}''$, found for a gage of $3\frac{3}{4} = 2\frac{1}{8} + 2\frac{1}{8} - \frac{1}{2}$. If these rivets are counted in the same right section, the net area is 28.00 sq. in. $= 2(21 - 5 \times 1)\frac{9}{16} + 2(3.25 - 2 \times 1 \times \frac{1}{2}) + 2(3.25 - 1 \times \frac{1}{2})$. The difference in strength between this section and the main section is only 33,900# = (30.12 − 28.00)16,000; this does not exceed 60,100, so the arrangement of rivets is satisfactory. Had this value been slightly in excess of the strength of the ten rivets, the net area should be found more accurately (preceding paragraph) before any change were made in the arrangement of rivets. In like manner, the

net section near the ends of a member may be reduced by changing the rivet stagger instead of the number of rivets. The minimum stagger from the diagram should be used for one or two spaces from the edge of the plate where the stress is maximum (i.e., towards the center of the member), but the remaining spaces may be reduced. As before, the net section at any point must be sufficient to carry that portion of the total stress that is not already transmitted by rivets.

COMPRESSION MEMBERS

1. The design of a compression member is an **indirect process** in which the form and the size of cross section are assumed, and the corresponding safe load is compared with the required stress. The result of the first trial furnishes a guide for the second assumption. The experienced designer can usually approximate the final section in his first assumption. Men who have considerable designing to do are equipped with more or less exhaustive tables of safe loads of members of different cross sections for different lengths.* Tables for struts composed of one or two angles are given on pages 330 and 331. No such table should be used, however, until the underlying principles are clearly understood.

2. **The strength** or safe load of a compression member is found by multiplying the unit stress by the gross area of cross section, without deducting for holes which are to be filled by rivets (page 206:4). The unit stress is not a fixed amount as for tension members, but it varies with the length of a member and with the form and area of its cross section. A long member is more liable to bend or buckle than a short one of the same cross section. When the area of cross section is distributed, it is more effective in resisting compression than when the same amount of material is compacted (page 206:3). The unit stress allowed in the design of a compression member is determined from a **"compression formula,"** "column formula," or "reduction formula." These formulas are of two general types, but both depend upon the "ratio of slenderness" $\frac{l}{r}$, in which l = the unsupported length of member and r = the least radius of gyration (see below), both in inches. One type is the "straight line formula," as for example unit stress = $16{,}000 - 70\frac{l}{r}$, which is the equation of a straight line. The other type is the "Rankine (or Gordon) formula," as for example, unit stress = $\dfrac{12{,}500}{1 + \dfrac{l^2}{36{,}000r^2}}$, which is the equation of a curve. Most specifications and building laws require the use of a formula of one of these types, but in many of them different numerical values are inserted in place of the 16,000 and the 70, or the 12,500 and the 36,000. The Rankine formula is used less commonly than formerly, because the straight line formula is more easily applied and gives results which are quite as satisfactory. The formula most commonly used is undoubtedly the $16{,}000 - 70\frac{l}{r}$, with a maximum value of 14,000, which is recommended by the American Railway Engineering Association; this has been widely adopted. The unit stresses obtained from this formula for different values of l and r are tabulated on page 328.

3. **The radius of gyration** is the term applied to the expression $\sqrt{\frac{I}{a}}$, or $I = ar^2$, in which a = the gross area of cross section, and I = the corresponding moment of inertia. The unit stress for any compression member without intermediate support is determined by the *least* radius of gyration, which is found from the least moment of inertia. Because of the rectangular construction of most structural steel members, the least moment of inertia is found about one of two perpendicular axes (except for a single angle). Often the axis about which the moment of inertia is least may be selected by inspection; otherwise, the moments about both axes must be found and compared. Thus for example, the moment of inertia and the corresponding radius of gyration of a rectangle are obviously less about an axis parallel to the longer side than about an axis at right angles to it. Accordingly, a rectangular member would buckle first in a direction at right angles to this axis, as is apparent from the manner in which an ordinary yardstick bends when compressed. The least radius of gyration of a single angle is found about a diagonal

* See also Ketchum's "Structural Engineers' Handbook," or Sample's "Properties of Steel Sections," McGraw-Hill Book Co., Inc., New York; also the handbooks of the different steel manufacturers.

axis, as shown in the tables on pages 325 and 326. The radii of gyration for members composed of two angles are given on page 329.

1. **The moment of inertia** about any axis of a cross section composed of several parts is found by combining the moments of inertia of the component parts about the *same* axis, even though the parts are on opposite sides of the axes. The moment of inertia I_c of any component part about an axis through its own center of gravity is found from a table; the moment of inertia I_A about any parallel axis AA is found by adding to this moment of inertia the product of the area of the component part by the square of the perpendicular distance between the axes, or $I_A = I_c + ax^2$. The moments of inertia and radii of gyration about perpendicular axes through their own centers of gravity are given in the tables for the following sections:* I-beams, pages 322 and 324; channels, page 323; angles, pages 325 and 326. The moments of inertia of plates about the axis perpendicular to the longer dimension are given on page 320. Most of the values used in the design of simple steel members are included in this table. Other values may be found by proportion or from the expression $\frac{bd^3}{12}$, in which d is the dimension at right angles to the axis. In most designs the moment of inertia of a steel plate about an axis through the center of gravity *parallel to the longer dimension* is negligible, but when this moment is transferred to a *parallel axis*, the product of the area by the square of the distance will be considerable. The distances from the centers of gravity to the back of channels and angles are also given on the corresponding pages; these distances may be used in finding the perpendicular distances between parallel axes. Care should be taken to choose the values for the proper axis. For angles, the axis parallel to the longer leg is marked L–L, while that parallel to the shorter leg is marked S–S. The diagonal axis about which the I and r are minimum is marked M–M. The sub-letters L, S, and M are used to distinguish the corresponding values. It is often most convenient to find the moment of inertia I_A of an unsymmetrical section about an axis AA through the center of gravity of one or more of its component parts (e.g., the center of webs), and then to transfer this moment to a parallel axis through the center of gravity of the whole section, by *subtracting* the product of the total area by the square of the eccentricity (i.e., the distance between the two axes), or $I_c = I_A - ae^2$. In transferring moments of inertia from one axis to another there is often some confusion as to whether to add or subtract the product of the area by the distance between axes. It should be remembered that for any given area the moment of inertia is least about an axis through the center of gravity of that area. If this moment is transferred to a parallel axis it should be increased. Conversely if the moment about a parallel axis is known, the moment about the center of gravity may be found by subtraction. The **eccentricity** of an unsymmetrical section may be found by equating the moments of areas. If a thin slice were cut from a member it could be balanced on a thin support along the axis CC through the center of gravity. If the support were placed along any other axis AA about which the moment of inertia I_A is known and from which the eccentricity is to be measured, the slice would tend to rotate about the support. This tendency is the same whether the section is considered as a whole or whether the component parts are considered separately. The product of the whole area by the eccentricity should equal the algebraic sum of the products of every component area by the distance from its center of gravity to the axis AA. Note that the moments of the areas on opposite sides of the axis AA have opposite signs; when the algebraic sum is zero the eccentricity must be zero. Some of the quantities are used in finding both the eccentricity and the moments of inertia, and the computation may be simplified accordingly.

2. **The forms of members** depend upon so many practical considerations that they cannot be discussed here. Single and double angles are commonly used for light compression members. Larger sections are composed of channels or angles with or without plates. The component parts of any member must be held in the proper relative position by stitch rivets, tie plates, lattice bars, or continuous plates in order to properly distribute the stress. Otherwise each component part is free to buckle independently, and the strength of the member is no greater than the combined strength of the component parts taken singly. The ratio of slenderness for any part of a member determined by the distance between tie plates or lattice bars must not exceed the ratio of slenderness for the

* For special rolled column sections in the form of the letter H see the handbook of the Bethlehem Steel Company or the Carnegie Steel Company.

entire member determined by its full length. This is usually provided for without special investigation by the usual method of spacing stitch rivets (page 69 : 4), or tie plates and lattice bars (page 70 : 1). The areas of continuous plates may be included in the effective cross section, but tie plates and lattice bars are not considered as part of the main section. The arrangement and the spacing of the component parts of a member have their effect upon the strength. To illustrate, let us consider the strength of two angles with unequal legs. If the member receives no intermediate support, the longer legs of the angles are placed together. This arrangement gives better distribution of metal because the radii of gyration about perpendicular axes are kept more nearly equal, and the least radius is larger than if the angles were placed with their shorter legs together. This is the usual arrangement for struts, bracing, and web members of latticed girders and roof trusses. In determining the strength of a member with intermediate support, such as a chord member of a latticed girder or light roof truss which is supported by web members, there are two different lengths to be considered. The unit stress is found from the greatest ratio of slenderness $\frac{l}{r}$, and not necessarily from the least radius of gyration. Since the length between lateral supports is usually greater than between panel points, the corresponding radius of gyration should also be greater in order to keep the ratios of slenderness more nearly equal, or at least so that the larger ratio is minimum. The angles of such members are thus usually placed with their shorter legs together. Each member should be analyzed in this manner. Certain practical limitations are often specified for the thicknesses of different component parts of compression members. Common examples are (*a*) that the thickness of web plates must not be less than $\frac{1}{30}$ the distance between the lines of rivets which connect the plates to the angles; (*b*) that the thickness of cover plates must not be less than $\frac{1}{40}$ the distance between the lines of rivets which connect them to the angles or channels; and (*c*) that the thickness of the angles without cover plates must not be less than $\frac{1}{12}$ of the outstanding leg of one angle.

1. The following **typical problems** illustrate the design or the investigation of some of the more common compression members.

First Problem. — Design a single strut 10 feet long to carry a stress of 50,000# at a unit stress of $16{,}000 - 70\frac{l}{r}$. The table of safe loads on page 330 is based upon this unit stress, so it may be used directly. Under the column headed 10 feet is found the stress 51,000 opposite a $6 \times 6 \times \frac{1}{2}$, which would be used. If such a table were not available, a table of properties of angles (page 325) must be used. Unless the designer is guided by experience or otherwise, he must assume a section and then investigate it. He may approximate the section roughly by dividing the given stress by a unit stress of 10,000#/sq. in. In this problem the required area is approximately 5 sq. in., so a $6 \times 6 \times \frac{7}{16}$ is investigated. The least radius of gyration of a single angle about a diagonal axis is 1.19 in. The unit stress is 8,940#/sq. in. $= 16{,}000 - \frac{70 \times 10 \times 12}{1.19}$; which multiplied by the area 5.06 sq. in. gives a safe load of 45,200#. This is too small, so the next larger size is tried in a similar manner and found sufficient, the safe load being 51,100#.

Second Problem. — Find the thickness of two $5 \times 3\frac{1}{2}$ angles to carry a stress of 75,000# at $16{,}000 - 70\frac{l}{r}$ pounds per square inch. The angles are 11 feet long and connect to a $\frac{1}{2}''$ gusset plate placed between them. If no other member is connected to this member in such a way as to give intermediate support to prevent its buckling in one direction, the longer legs of the angles will be connected to the plate and hence be parallel and $\frac{1}{2}''$ apart. The radii of gyration about both axes appear in the middle table on page 329. The approximate area of two angles is 7.5 sq. in. $= 75{,}000 \div 10{,}000$. From the areas of two angles in the above table this falls between $\frac{7}{16}$ and $\frac{1}{2}$ inch in thickness. The latter would have a safe load of $80{,}000\# = 8.00\left(16{,}000 - \frac{70 \times 11 \times 12}{1.54}\right)$, the former only 70,000#, hence 2 Ls $5 \times 3\frac{1}{2} \times \frac{1}{2}$ would be used. If the angles were $\frac{3}{8}''$ apart the safe load could be taken directly from the table on page 331. If neither of these tables were available the radii of gyration would have to be found from the properties of angles (page 325). The radius about an axis parallel to the shorter legs is the same for one or two angles because the axis through the center of gravity of the member

passes through the centers of gravity of each angle; thus both the moment of inertia and the area are doubled and the radius remains unchanged. The moment of inertia about the other axis is 18.90 in^4 $= 2[4.05 + 4.00(0.25 + 0.91)^2]$ and the corresponding radius is 1.54 in. $= \sqrt{18.90 \div (2 \times 4.00)}$.

Third Problem. — Find the safe load of a 20-foot column composed of two $12 \times \frac{1}{2}$ plates and two 10″ ⊔ 15#, 6″ back to back, as shown in Fig. 214 (*a*). The allowed unit stress $= \dfrac{12{,}500}{1 + \dfrac{l^2}{36{,}000r^2}}$.

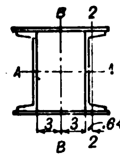


Fig. 214 (*a*).

6.00 sq. in. = area of one $12 \times \frac{1}{2}$ (by inspection, or from page 321)
72.0 in^4 = I " " $12 \times \frac{1}{2}$ (page 320)
4.46 sq. in. = area " " 10″ ⊔ 15#
66.9 in^4 = I " " 10″ ⊔ 15# about axis AA
2.3 in^4 = " " " 10″ ⊔ 15# " " BB
0.64 in. = dist. from b. of web to c. of g.
(page 323)

133.8 = 2×66.9 = I of 2 ⊔ about AA
330.7 = $2 \times 6.00(5.00 + 0.25)^2$ = I of 2 Pls. about AA (neglecting the I about axis through c. of g. of Pl.)
464.5 = total I about AA

122.8 = $2(2.3 + 4.46 \times 3.64^2)$ = I of 2 ⊔ about BB
144.0 = 2×72.0 = I of 2 Pls. about BB
266.8 = total I about BB

12.7 in^2 = $266.8 \div 2(6.00 + 4.46)$ = the least r^2

$$232{,}000\# = \frac{20.92 \times 12{,}500}{1 + \dfrac{(20 \times 12)^2}{36{,}000 \times 12.7}} = \text{the total safe load.}$$

Note that when the compression formula contains r^2 it is unnecessary to find r.

Fourth Problem.— Find the least radius of gyration for a top-chord section composed of 2 web plates $18 \times \frac{1}{2}$, 1 cover plate $21 \times \frac{1}{2}$, two top angles $3 \times 3 \times \frac{3}{8}$, and 2 bottom angles $4 \times 3 \times \frac{1}{2}$, arranged as shown in Fig. 214 (*b*). It is convenient to select all the necessary areas, moments of inertia, and distances to centers of gravity from the tables at the outset. The distances may be recorded directly upon the sketch.

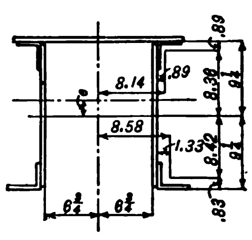


Fig. 214 (*b*).

9.00 sq. in. = area of one Pl. $18 \times \frac{1}{2}$
10.50 " " = " " " " $21 \times \frac{1}{2}$
2.11 " " = " " " L $3 \times 3 \times \frac{3}{8}$
3.25 " " = " " " " $4 \times 3 \times \frac{1}{2}$
1.8 in^4 = I " " " $3 \times 3 \times \frac{3}{8}$ about axis through its c. of g.
2.4 " = " " " " $4 \times 3 \times \frac{1}{2}$ " hor. axis through its c. of g.
5.0 " = " " " " $4 \times 3 \times \frac{1}{2}$ " vert. " " " " " "
243.0 " = " " " Pl. $18 \times \frac{1}{2}$ " hor. " " " " " "
385.9 " = " " " " $21 \times \frac{1}{2}$ " vert. " " " " " "
39.22 sq. in. = $2(9.00 + 2.11 + 3.25) + 10.50$ = total area

99.8 in^3 = $10.50 \times (9.25 + 0.25)$ = moment of cover plate
35.3 " = $2 \times 2.11 \times 8.36$ = " " 2 top Ls
135.1 " = sum
54.7 " = $2 \times 3.25 \times 8.42$ = " " 2 bottom Ls
80.4 " = algebraic sum of moments about centers of webs.
2.05 in. = $80.4 \div 39.2 = e$ = eccentricity.

486 in^4 = 2×243.0 = I of webs about centers of webs
948 " = 10.50×9.50^2 or 99.8×9.50 = I of cover plate
295 " = $2(1.8 + 2.11 \times 8.36^2)$ or $3.6 + 35.3 \times 8.36$ = I of top Ls
461 " = $2(2.4 + 3.25 \times 8.42^2)$ or $4.8 + 54.7 \times 8.42$ = I of bottom Ls
2190 " = total I about axis through centers of webs.
165 " = 39.22×2.05^2 or 80.4×2.05
2205 " = total I about horizontal axis through c. of g.

386 in⁴ = I of cover plate about vertical axis
882 " = $2 \times 9.00 \times (6.75 + 0.25)^2$ = I of webs
283 " = $2(1.8 + 2.11 \times 8.14^2)$ = I of top **L**s
483 " = $2(2.4 + 3.25 \times 8.58^2)$ = I of bottom **L**s
2034 " = total I about vertical axis through c. of g.
7.2 in. = $\sqrt{2025 \div 39.2}$ = least radius of gyration.

The table of squares (page 332) may be used to advantage in a problem which involves squares or square roots.

MEMBERS WHICH RESIST BENDING AND DIRECT STRESS

1. Tension or compression members, such as the top or bottom chords of roof trusses or the end posts of bridge trusses, are sometimes subjected to transverse bending as well as direct axial stresses. They must be designed so that the combined stresses on the extreme fibers do not exceed the allowed unit stresses determined from the compression formulas.* The unit stress in direct tension is found by dividing the total tensile stress by the net area of cross section. The unit stress in direct compression is found by dividing the total compressive stress by the gross area of cross section. These areas should be taken near the point of maximum bending. The unit stress in the extreme fiber due to bending is found by equating the bending moment to the resisting moment $\frac{fI}{c}$ and solving for f. In an unsymmetrical member the value c in a tension member should be taken from the neutral axis to the extreme fiber in tension, while in a compression member it should be taken from the neutral axis to the extreme fiber in compression. Two allowed unit stresses should be considered, one with the radius of gyration about an axis perpendicular to the direction of bending, the other with the radius about an axis parallel to the direction of bending. The former should not be exceeded by the combined unit stresses due to bending and direct stress, but the latter should not be exceeded by the direct unit stress alone.

Illustrative Problem. — Design a horizontal compression member 8 feet long which will support a load of 2000# at its center. The direct compression is 75,000# and the member is composed of two 6 × 4 angles, ½″ b. to b. 4000# ft. = $\frac{2000}{2} \times 4 = M_B$.

Assume 2 **L**s 6 × 4 × ½, area = 9.50, distance from center of gravity to back of shorter leg = distance from neutral axis to extreme fiber in compression = c = 1.99, I = 34.8, and r = 1.91 about the axis perpendicular to the direction of bending.

7,900#/sq. in. = 75,000 ÷ 9.50 = direct unit stress
2,740 " " = $\frac{4000 \times 12 \times 1.99}{34.8}$ = unit stress due to bending
10,640 " " = total unit stress
12,480 " " = $16{,}000 - \frac{70 \times 8 \times 12}{1.91}$ = allowed unit stress which is considerably greater than 10,640.

Assume 2 **L**s 6 × 4 × $\frac{7}{16}$, area = 8.36, c = 1.96, I = 31.0, r = 1.92
8,970#/sq. in. = 75,000 ÷ 8.36 = direct unit stress
3,040 " " " = $\frac{4000 \times 12 \times 1.96}{31.0}$ = unit stress due to bending
12,010 " " " = total unit stress
12,500 " " " = $16{,}000 - \frac{70 \times 8 \times 12}{1.92}$ = allowed unit stress
12,000 " " " = $16{,}000 - \frac{70 \times 8 \times 12}{1.68}$ = allowed unit stress for compression alone.

Since 12,010 is reasonably close to 12,500 (⅜″ angles would be too small) and since 8,970 does not exceed 12,000, 2 **L**s 6 × 4 × $\frac{7}{16}$ would be used.

* Some designers use different unit stresses for bending and for direct stress and combine the resulting areas; see Johnson-Bryan-Turneaure's "Framed Structures," Vol. III, John Wiley and Sons, Inc., New York.

LATTICE BARS AND TIE PLATES

1. **The design** of lattice bars and tie plates or batten plates for compression members cannot be theoretically developed. Certain minimum sizes based upon the experience of many years are demanded by the specifications in common use. These requirements often determine the sizes to be used, but the lattice bars must also be large enough to carry the stresses found by the method explained below. This method of design gives safe results for all members except those of very large structures which should receive more careful consideration. Some recent column tests seem to indicate that this method results in lattice bars about twice as strong as necessary.

2. **A tie plate** should be placed in the plane of every system of lattice bars as near each end of the member as possible, and wherever the lattice system is interrupted. The plates at the ends should extend longitudinally at least as far as the transverse distance between the lines of rivets which attach them to the member. Intermediate plates need be only one-half as large. The rivets in tie plates are usually spaced about 3″ apart. The thickness of tie plates should not be less than $\frac{1}{50}$ of the distance between the rivet lines mentioned above. A plate provided for another purpose may serve also as a tie plate even though it is not placed in position before erection. Tie plates with or without lattice bars are used on tension members to distribute the stresses, although they do not need to be designed to resist buckling.

3. The minimum widths of **lattice bars** are commonly specified as follows: $2\frac{1}{2}''$ for $\frac{7}{8}''$ rivets, $2\frac{1}{4}''$ for $\frac{3}{4}''$ rivets, and 2″ for $\frac{5}{8}''$ rivets. $\frac{7}{8}''$ rivets are used for latticing flanges $3\frac{1}{2}''$ wide or over. One rivet is generally used at each end unless the flanges exceed 5″ in width, when two are used. Double lattice bars are used when the transverse distance between rivet lines exceeds 1′ 3″. The minimum thickness of single lattice bars is $\frac{1}{40}$ of the length from center to center of end holes; similarly the minimum thickness of double lattice bars is $\frac{1}{60}$ of the same length. The table on page 315 shows the maximum lengths for different thicknesses for these ratios and for others which are sometimes specified.

4. **Method of Design.** — Compression members which are designed for a unit stress of $16{,}000 - 70\frac{l}{r}$ could be designed for 16,000 the same as a tension member were it not for the tendency to buckle. It may be reasoned that the term $70\frac{l}{r}$ represents the stress in the extreme fiber due to this tendency to buckle. It may fairly be assumed that in effect this is equivalent to the fiber stress caused by transverse bending forces uniformly distributed, the member acting as a beam. The corresponding shearing stresses are taken either by continuous plates (as in a beam or girder) or by lattice bars (as the diagonals in a latticed girder). The stress in each bar may be found accordingly as follows:

$M_B = \frac{UL^2}{8}$ (page 188 : 2); if we let $R = \frac{UL}{2}$ = the reaction or maximum shear we have $M_B = \frac{RL}{4}$ whence $m_B = \frac{Rl}{4}$

Also $I = ar^2$ (page 211: 3), and $f = 70\frac{l}{r}$ (from above).

Substituting these values in $m_B = \frac{fI}{c}$ (page 199: 2), we have $R = \frac{280\,ar}{c}$, in which a = the total area of cross section, r = the radius of gyration about an axis perpendicular to the plane of the lattice bars, and c = the distance from this neutral axis to the extreme fiber. This shear is the transverse component of the stress in the lattice bar and from it the stress can be determined by multiplying by the cosecant of the angle which the lattice bar makes with the longitudinal axis of the member. If this angle is 45°, the stress in the bar is $\frac{396\,ar}{c}$. If a member is latticed in two planes, the bars in each plane are stressed only one half the above amount. Similarly, if a continuous plate is used on one side and lattice bars on the other as in chord members, the plate and the system of bars may each be considered to take one-half. Since lattice bars resist both tension and compression they must be strong enough for either. They are designed for compression at $16{,}000 - 70\frac{l}{r}$, for this determines the size.

Illustrative Problem. — Design the lattice bars for the top-chord section of the fourth problem on page 214. Since a cover plate is used on top, the bottom lattice bars need be designed for only half the bending. From Fig. 214 (*b*) the distance between rivet lines is obviously more than 1′ 3″, so double latticing is required, and therefore the angle of inclination is 45°. The maximum stress in one bar is $\frac{1}{2} \times \frac{1}{2} \times \frac{396ar}{c}$ in which $a = 39.2$, $r = 7.2 = \sqrt{\frac{2034}{39.2}}$, and $c = 11.25 = 6.75 + 0.50 + 4.00$. Substituting these values and solving, we find the stress in one bar $= 2480\#$. $\frac{7}{8}$″ rivets should be used since the 4″ angle exceeds $3\frac{1}{2}$″, and bars at least $2\frac{1}{2}$″ wide must be used. The length of each bar from center to center of end rivets is $28'' = 1.41 \times 2(6.75 + 0.50 + 2.50)$, and the minimum thickness is $\frac{1}{2} = \frac{28}{56}$. The area of a $2\frac{1}{2} \times \frac{1}{2}$ is 1.25, and the radius of gyration is $0.14 = \sqrt{\frac{2.5 \times 0.5^3}{12 \times 2.5 \times 0.5}}$, whence the safe load is $2500\# = 1.25\left(16{,}000 - \frac{70 \times 28}{0.14}\right)$. This is greater than the required stress 2480, so the bars are satisfactory.

CHAPTER XXXIII

THE DESIGN OF PLATE GIRDERS

SYNOPSIS: Three methods of designing the main cross section of a girder are presented and compared. The discussion of details, such as the lengths of cover plates, the spacing of flange rivets, and the sizes of stiffening angles and splices is left for subsequent chapters.

1. Plate girders are designed to meet a great variety of conditions, as explained on page 95 : 1. **The common forms** of cross section are shown in Fig. 95, the majority of girders being of one of the first two forms. In order to simplify the phraseology, most of this chapter has been written to apply to horizontal girders with vertical loads, each flange being composed of two angles, with or without cover plates.

2. **Analysis of Forces.** — The external forces which act upon a plate girder must satisfy the three equations of equilibrium (page 183:2). If the girder is cut by an imaginary plane, the external forces which act on either segment are not in equilibrium by themselves, but they are held in equilibrium by the internal forces or "stresses" acting in the fibers which are cut by the section plane. The vertical components of these forces are shearing stresses; these are resisted by the web, as discussed in Chapters XXXIX and XL, pages 266 and 270. The horizontal components of these internal forces must satisfy the H equation of equilibrium provided the external forces are all vertical; thus the sum of all tensile stresses (which tend to elongate the girder) must equal the sum of all compressive stresses (which tend to shorten the girder) as in a beam. The sum of the moments of the external forces which act on the segment is the bending moment, and the sum of the moments of the internal forces cut by the section plane is the resisting moment; these two moments must be equal in order to satisfy the M equation of equilibrium. If the point of moments is taken in the section plane the moment of the vertical components of the internal forces is zero, and only the horizontal components need be considered. The section plane is taken at the point of maximum bending moment and the girder is designed to furnish the proper resisting moment by one of the methods described in this chapter.

3. **The depth** of a plate girder is often predetermined by specific requirements. Within practical limits, flanges may be designed for any depth of girder; as the depth is increased the flanges become lighter, and conversely. The most economical depth is from one-seventh of the length for short spans to one-twelfth the length for long spans; in the absence of other data an average value of one-tenth the length may be chosen; the maximum depth is limited to about 10′-6″ by the overhead clearance available during shipment. The depth of the web plate is usually made a multiple of 2″, and preferably a multiple of 6″; the depth from back to back of flange angles is usually $\frac{1}{4}$″ or $\frac{1}{2}$″ greater than the depth of the web (page 95 : 3).

4. The thickness of **the web plate** must be such that the strength of the web is sufficient to transmit the shearing stresses, as explained on page 266: 3. The web plate must also be thick enough to furnish proper bearing for the flange rivets, as explained on page 255: 2. The usual values are from $\frac{5}{16}$″ to $\frac{5}{8}$″ (page 266: 3) with a minimum value equal to $\frac{1}{160}$ of the vertical clear distance between flange angles. Usually a value is assumed with due regard to this minimum, then the flanges of the girder are designed, and finally the strength of the web plate is investigated. If necessary, these steps may be repeated until a satisfactory solution is obtained.

1. **The flanges** of comparatively light girders are composed of two angles each. Cover plates are added to the flanges of heavier girders to provide additional flange area where needed; the cover plates, unlike the angles, need not extend the full length of the girder, but they may be cut off at points beyond which the remaining area is sufficient to carry the reduced flange stress, as explained in Chapter XXXVIII, page 259. It is not practical to use cover plates unless the outstanding legs of the flange angles are at least 5 or 6 inches; the cost of using heavier angles would be less than the cost of using cover plates on account of the extra punching and riveting. Additional angles or vertical plates may be used in the heavier girders, as shown in Fig. 95, *c*, *d*.

2. **Flange angles** with unequal legs should be used whenever practicable; angles with equal legs less than 6 inches are seldom used. 6×6 angles are used when $6 \times 4 \times \frac{3}{4}$ angles are not large enough, or when the flange rivets in the web legs must be staggered to meet the requirements of Chapter XXXVII, page 241. 8×8 angles are reserved for very heavy girders; they are not used without cover plates. When unequal legs are used, the shorter leg is riveted to the web plate and the longer leg is outstanding. In this position the center of gravity of the flange is nearer the back of the angle and hence farther from the neutral axis; the resisting moment is thus made correspondingly greater (why?). The lateral stiffness is also increased by placing the longer legs horizontally.

3. **The cover plates** are made wide enough to fully cover the angle; plates of commercial widths will usually project beyond the angles. 18″ plates (or 17) are used with 8″ angles, 14″ plates (or 13) with 6″ angles, and 12″ (or 11) plates with 5″ angles. It is convenient to find the total thickness of cover plates as if there were only one plate on each flange; this total thickness may then be subdivided into the proper number of plates. No plate should be thicker than $\frac{3}{4}''$ nor less than the minimum thickness of metal allowed (usually $\frac{3}{8}''$). Usually the number of plates used is the smallest which will meet these requirements so that the cost of handling and punching extra plates may be saved. Full-sized holes cannot be punched satisfactorily in metal thicker than $\frac{3}{4}''$; they must be either drilled or sub-punched and reamed (page 30 : 2). The cover plates should be made approximately of equal thickness, but they may differ by $\frac{1}{16}''$ when the total thickness is not an exact multiple of the number of plates. When plates of different thickness are used, the thicker plates should be nearer the angles.

4. **Distribution of Area.** — The net of the flange angles should preferably be as large as the net area of the cover plates. In other words, the net area of the angles should be 50 % of the area which remains after the portion (if any) of the web plate counted as flange area is deducted from the total net area required. This requirement is often specified in order to overcome the tendency of a few designers to put considerably more than 50 % of the area into the cover plates to save metal; this tendency might result in angles which are too weak to transmit the corresponding stress from the web plate to the cover plates.

The author has seen recently a flagrant violation of this specification in an existing main-line railroad bridge in which cover plates about 6″ thick are connected to the web by $6 \times 3\frac{1}{2} \times \frac{1}{2}$ flange angles.

In some of the very heavy girders it is impossible to obtain angles which are large enough to furnish 50 % of the area mentioned, but the specifications provide for this contingency by allowing "the largest size of angle" to be used. In case no clause appears in the adopted specifications * regarding the relative distribution of area, it is the common practice to make the area of the angles 50 % of the net area remaining after $\frac{1}{8}$ (or other portion) of the area of the web plate is deducted, *provided* the angles are no thicker than $\frac{3}{4}''$. Thus, $6 \times 6 \times \frac{3}{4}$ angles are often used even though the cover plates are of somewhat greater area; this is done so that the rivet holes in the angles may be punched (see preceding paragraph).

5. **The Compression Flange.** — It is desirable from the standpoint of fabrication to make both flanges of a plate girder alike. Unless the top flange is braced transversely to prevent buckling it may have to be made of a different cross section. The flange stress is the same in the compression flange as it is in the tension flange as will be seen later (page 221 : 2). The allowed unit stress is less in compression than in tension, but the effective area is greater because no deduction need be made for rivet holes. The tension flange is usually designed, and the

* For example, the "Specifications for Steel Railway Bridges," of the American Railway Engineering Association, Chicago.

compression flange is made like it unless the compression flange is found not to have the proper lateral stiffness; the compression flange should never be made of smaller gross area than the tension flange. The compression flanges of bridge girders are usually braced at intervals either by lateral bracing (Fig. 142) or by gussets at the floor beam connections (Fig. 99); in the majority of cases the two flanges may be made alike. The compression flanges of crane-runway girders are sometimes latticed to other girders (page 112 : 6), in which case the flanges may be made alike; otherwise the top flange must often be made heavier or wider, as shown in Fig. 95 (*e*) and (*f*). After the tension flange of a girder is designed, the strength of a like compression flange should be investigated. The allowed unit stress is based upon the ratio of the width of the flange to the unsupported length. The specifications recommended by the American Railway Engineering Association state that the unit stress per square inch of *gross* area of the compression flange must not exceed $16{,}000 - 200\frac{l}{b}$ when angles alone or angles with cover plates are used, or $16{,}000 - 150\frac{l}{b}$ when angles with channel covers are used; b = the extreme width of flange in inches, and l = the distance in inches between lateral supports. If the compression flange is not braced, l = the full length of the girder. The thickness of flange angles used without cover plates should not be less than one-twelfth the length of each outstanding leg.

1. **Three Methods of Design.** — The flanges of a plate girder may be designed by any one of three methods, viz.: Case A, in which the flange angles and cover plates are assumed to resist the entire flange stress due to bending moment; Case B, in which the resisting moment of the web plate is considered; and Case C, in which the moment of inertia of the net cross section is used in much the same manner as in the design of beams. The method of Case C is given as an alternate method in the more modern specifications, but it is not recommended for general use on account of the somewhat indirect and laborious calculation involved. It is practical only when exhaustive tables are accessible, which give the moments of inertia or the section moduli of all types and sizes of girders, for different sizes of rivet holes. This method is an application of the general formula for flexure, $m_B = \frac{fI}{c}$ (page 199 : 2) in which I = the moment of inertia of the *net* cross section. Theoretically the neutral axis is not at the center of the web because the tension half of the girder is weakened by the rivet holes, while the compression half is not. This refinement is not adopted ordinarily, but for convenience the upper half is considered like the lower half; that is, the moments of inertia of the holes in both flanges and in the whole web are deducted from the moment of inertia of the cross section (page 212 : 1). The method of Case B is meeting with most favor at the present time, and has been adopted quite generally. The results are obtained more readily by means of this method than by the more precise method of Case C, and they are very nearly the same. The assumption is made that the flange stress is uniformly distributed throughout the whole flange, the resultant acting at the center of gravity of the flange. The effective depth is therefore the distance between the centers of gravity of the flanges; it cannot be ascertained definitely until the sizes of the angles and the cover plates are known. This necessitates a trial design, which makes the method somewhat more complex than that of Case A. Case B is treated more fully on page 223 : 1. The method of Case A is comparatively simple in application, but it is not well adapted to the design of all girders for it would result in a waste of metal in some of the heavier girders. In this method the resisting moment of the web plate is neglected, but this is compensated for, in part, by the use of an increased depth (usually the depth of the web plate). The use of this method gives safe results; for girders without cover plates, or for girders with 6×6 angles and a single cover plate in each flange, the results are approximately the same as those obtained by the method of Case B or C. Until the effect of impact from moving loads is better understood so that different specifications are made more nearly uniform, and until there is a closer agreement between actual loads and those assumed, the method of Case A should give consistently good results for the lighter girders, particularly, those mentioned in the preceding sentence. Case A is treated more fully below.

2. **The degree of accuracy** to be used in computation depends largely upon the precision of the given data. Usually loads are expressed to the

9 : 3); this resisting moment is $\frac{1}{6}ftd_w^2$, where t = the thickness
ɘb plate (i.e., the width of the beam) and d_w = the depth of the
e. In this method the effective depth, and the thickness and
ı of the web are expressed in inches, so the resisting moment in
ches must be equated to the maximum bending moment in
ches, thus: $fa'd_g + \frac{1}{6}ftd_w^2 = m_B$. Instead of using the resisting
of the web plate in this form it is found to be more convenient
it by the effective depth and by the unit stress to give a quantity
n be combined with the net flange area of the angles and the
ιtes to give the total net area, thus: $a' + \frac{1}{6}td_w\left(\frac{d_w}{d_g}\right) = a = \frac{m_B}{fd_g}$.
xpression the full gross area of the web plate is used, no allowance
ιde for rivet holes; but there will be holes in the web for the
vets, and usually for rivets in stiffening angles (page 266 : 2).
ay be no stiffening angles exactly at the point of maximum bend-
ιent, nevertheless they may be placed where the moment is only
less. The position of the stiffening angles and the spacing of
ınnot be determined until after the girder is designed, so it is
cal to attempt to find the actual net area. Furthermore, the
of the web plate counted as net area is small compared to the
of the angles and cover plates, so a slight variation will have
ιtively little effect. For convenience, a general method is used
rovides for rivets under average conditions. Thus if we consider
s to be spaced 4 inches center to center, it means that a 1″
leducted every 4 inches, leaving 3 inches of metal between; thus
area of the web plate is $\frac{3}{4}$ of the gross area. The effect of
to multiply the fraction $\frac{1}{6}$ in the expression given above by $\frac{3}{4}$
$\frac{1}{8}$. In view of the approximation resulting from the assump-
he size and the spacing of the rivets, it is consistent and on the
ıfety to call $\frac{d_w}{d_g}$ equal to unity. Our revised formula then becomes

$$a' + \tfrac{1}{8}td_w = a \qquad \text{or} \qquad a' = a - \tfrac{1}{8}td_w.$$

from the total net area required we subtract one-eighth of the
a of the web, leaving the net area to be taken by the angles and
ιtes.

1. **The effective depth** is expressed to the nearest tenth of an inch. The distance from the center of gravity of the gross area of the flange angles and cover plates to the back of the angles is found,* then twice this distance is subtracted from the depth of the girder from back to back of angles to give the effective depth. The depth from back to back of angles is not the same as the depth of the web plate, but is usually $\frac{1}{2}$″ greater unless the upper edge of the web plate is unprotected from the weather when it is only $\frac{1}{4}$″ greater (see page 95 : 3). Since the effective depth cannot be determined accurately until the sizes of the angles and the cover plates are known, it is necessary to make a trial design with an assumed depth. No convenient rule can be given for selecting the proper depth to use, but an experienced designer is guided by the results of previous designs. In the absence of more definite information the following guides may be used. For girders without cover plates the problem is comparatively simple because the distance from the center of gravity to the backs of the angles does not vary greatly for different thicknesses unless the size of leg changes. Thus, if the size of the legs is determined or assumed, the distance from the center of gravity to the back of an angle of intermediate thickness may be taken from the table on page 325, and a close approximation of the effective depth may be obtained. If angles of unequal legs with cover plates, or if equal-legged angles with more than two cover plates are used, the trial depth should be taken equal to the depth of the web plate; but if equal-legged angles with only one or two cover plates are used, the depth should be about 1″ less than the depth of the web. From the sizes of the angles and cover plates found as a result of the trial design the corrected effective depth should be found to the nearest tenth of an inch. The bending moment due to the weight of the girder should be revised also, and a second solution of the problem should be made. If the sizes of the angles or cover plates differ from those found in the trial design the corresponding effective

* For convenience the point of moments is taken at the back of the angles. The difference between the product of the gross area of the angles by the distance to the center of gravity of the angles and the product of the gross area of the cover plates by one-half the total thickness of cover plates, is divided by the sum of these gross areas; the resultant distance is measured toward the angles or toward the cover plates according to which of the above products is larger.

depth should be determined; a third design is not necessary provided the corrected effective depth does not differ from the depth used in the second design by more than 0.1″ for girders less than 3 ft. deep nor by more than ¼″ for deeper girders.

1. **Illustrative Problem — Case B.** — Design the girder shown in Fig. 225, for a 60-foot single-track through railway bridge to support Cooper's E60 live load, using the specifications of the American Railway Engineering Association. We will use ⅞″ rivets, and assume a $72 \times \frac{7}{16}$ web plate (see page 218 : 3). The length is divided into four equal panels of 15 ft., and the width of the bridge from center to center of girders is 14′ 6″. The total dead load of the track is taken as 450#/ft., including the rails, the ties, the guard rails, the splice bars, the fastenings, etc. The design of the stringer and floor beam naturally precedes the design of the girder; we will assume that these have been designed and that the weights have been determined to be 150#/ft., for each stringer and 175#/ft. for each floor beam. The maximum concentrated live loads transmitted from the floor beams to this girder were found on page 196 : 2 to be 39,900#, 82,000#, and 52,100, respectively; to these should be added the corresponding impact allowance for the effects of moving loads, and also the dead loads, to give the total concentrated loads. The bending moment due to these concentrated loads should be increased by the bending moment due to the assumed weight of the girder itself which is uniformly distributed. The allowance for impact stresses is provided for in the specifications in the form of a percentage of the live load. In the A.R.E.A. specifications mentioned above this percentage is determined from the expression* $\frac{300}{L + 300}$, in which L is the length of track in feet which must be loaded to cause the maximum live-load strain in the member. In this problem the maximum bending moment in the girder will be found when the track is loaded for the entire length of the girder, hence the impact percentage is $\frac{300}{360}$. The solution follows:

$$6900\# = \left(\frac{450}{2} + 150\right)15 + 175 \times \frac{14.5}{2} = \text{dead load at each panel, due to track, stringers, and floor beam}$$

$$80{,}100\# = 39{,}900\left(1 + \frac{300}{360}\right) + 6900 = \text{total load at 1st panel point}$$

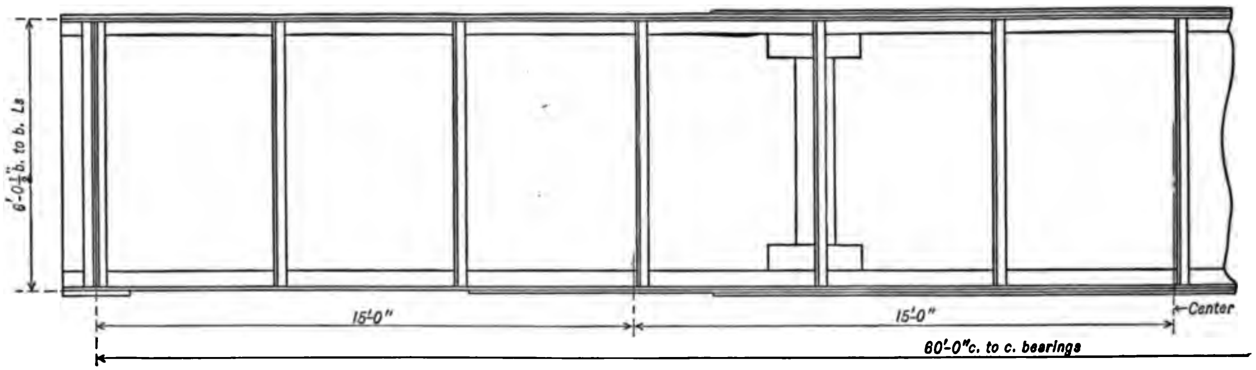


Fig. 225.

$$157{,}200\# = 82{,}000\left(1 + \frac{300}{360}\right) + 6900 = \text{'' '' '' 2nd '' ''}$$

$$102{,}400\# = 52{,}100\left(1 + \frac{300}{360}\right) + 6900 = \text{'' '' '' 3rd '' ''}$$

$$164{,}300\# = (80{,}100 \times 3 + 157{,}200 \times 2 + 102{,}400 \times 1) \div 4 = \text{left-hand reaction}$$

$$3{,}728{,}000\#\text{ft.} = 164{,}300 \times 30 - 80{,}100 \times 15 = M_B \text{ due to conc. loads (Fig. 251 }(a))\text{.}$$

$$370\#/\text{ft.} = 107 + 4 \times 29 + 2 \times 71 = \text{approx. weight of girder } (6 \times 6 \times \tfrac{3}{4}\ Ls \text{ and } 14 \times 1\tfrac{1}{2} \text{ cover plates assumed})$$

$$167{,}000\#\text{ft.} = \frac{370}{2} \times 30^2 = M_B \text{ due to weight of girder}$$

* A change to $\frac{30{,}000}{30{,}000 + L^2}$ has been recommended but it is meeting with opposition and it has not yet been adopted.

3,895,000#ft. = 3,728,000 + 167,000 = total M_B

40.6 sq. in. = $\dfrac{3{,}895{,}000 \times 12}{16{,}000 \times 72}$ = total net area required (d_g assumed 72″)

3.9 sq. in. = $\frac{1}{8} \times 72 \times \frac{7}{16} = \frac{1}{8}$ the gross area of the web

36.7 sq. in. = net area required in angles and cover plates

13.9 sq. in. = $2(8.44 - 2 \times 1 \times \frac{3}{4})$ = net area 2 Ls $6 \times 6 \times \frac{3}{4}$ (see page 303)

22.8 sq. in. = balance required in cover plates

$1\frac{15}{16}$″ = 1.90″ = $22.8 \div (14 - 2 \times 1)$ = thickness of 14″ cover plates

0.09″ = $\dfrac{2 \times 8.44 \times 1.78 - 27.13 \times 0.97}{16.88 + 27.13}$ = distance from back of angles to center of gravity (Fig. 226)

72.3″ = $72 + 0.5 - 2 \times 0.09$ = effective depth d_g

410#/ft. = $107 + 4 \times 29 + 2 \times 92$ = revised weight of girder

185,000# ft. = $\dfrac{410}{2} \times 30^2$ = revised M_B due to weight of girder

3,913,000#ft. = 3,728,000 + 185,000 = revised total M_B

40.6 sq. in. = $\dfrac{3{,}913{,}000 \times 12}{16{,}000 \times 72.3}$ = revised total net area

Use
- 2 Ls $6 \times 6 \times \frac{3}{4}$
- 1 Pl. $14 \times \frac{11}{16}$
- 2 Pls. $14 \times \frac{5}{8}$

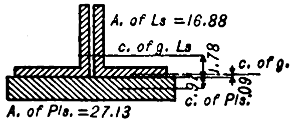


Fig. 226.

Note that the revised total area is the same as the first total, so no further revision is necessary; this is because the increase due to increased weight happened to compensate the decrease due to the larger depth. Had there been a difference in area, the balance (22.8) required in the cover plates could have been changed by a like amount and the corresponding thickness could have been found. For comments upon the arrangement see pages 185 : 1 and 208 : 3. For suggestions upon the solution see page 222 : 2. Subsequent steps in the complete design of this girder are given on pages, 226 : 1, 250 : 3, 262 : 1, 265 : 1, 269 : 1, and 273 : 1.

1. Railway bridges and viaducts are subject to **lateral forces** due to the effects of the rocking or "nosing" of the locomotives, and of the wind pressure both upon the structures themselves and upon passing trains. These lateral forces are often treated together. They act horizontally and they are resisted by the bracing systems. The effects of the train are most severe upon the "loaded chords," i.e., the top flange of stringers and deck girders, or the bottom flanges of through girders. The American Railway Engineering Association specifies a lateral force of 200#/ ft. on the unloaded chord, while on the loaded chord this is increased by 10% of the specified train load which follows the engines; both are considered as moving loads. The bending moment due to these lateral forces cannot be combined with the bending moment due to the vertical forces because they act in different planes; but the corresponding flange stresses or flange areas can be added. The lateral bracing forms the web system of a horizontal truss, the chords of which are the flanges of the girders. By the method of sections the stress in one of the chords at the center is found by dividing the sum of the moments of the external forces on one segment by the depth of the truss, i.e., the perpendicular distance between girders. The sum of the moments is equivalent to the bending moment of the uniformly distributed lateral forces, even though the forces are concentrated at the floorbeams. Since the maximum flange stress due to the vertical forces, and the maximum flange stress due to the horizontal forces are not likely to occur simultaneously, it is customary to neglect the latter unless the combined stress per square inch exceeds by more than 25% the unit stress allowed for vertical forces alone. When the combined stress does exceed this amount the flange should be strengthened. The effect of lateral forces upon the problem of page 225 : 1 should be considered as follows:

800#/ft. = $200 + 6000 \times 0.10$ = specified lateral force on bottom chord

360,000# ft. = $\dfrac{800}{2} \times 30^2 = M_B$ due to lateral force

1.6 sq. in. = $\dfrac{360{,}000}{16{,}000 \times 14.5}$ = corresponding net area required.

This is less than 25% of the net area 40.6 which is required by the vertical forces alone; hence the combined unit stress will not exceed $16{,}000 \times 1.25$, and no change in section need be made.

1. The method of designing heavy girders with **vertical flange plates, with four angles in each flange,** or with both, is similar to the method of Case B (page 223 : 3) only somewhat more complex. The effective depth is the distance between the centers of gravity of the gross area of the flange angles, the cover plates, and the vertical flange plates. One-eighth of the gross area of the web plate may be counted as flange area. It is assumed that only experienced designers will have occasion to design girders of this type, so that further comments would be out of place here.

2. Girders which support curved railroad tracks should be designed to resist **centrifugal forces.** These forces vary with the degree of curvature and with the weight and the velocity of the train. The forces are treated as uniformly distributed horizontal forces for which the horizontal bending moment and the corresponding flange stress or flange area must be found in the same manner as for lateral forces (see above). The flange should be designed for the sum of this area and the area found from the vertical forces, no increase in unit stress being allowed as for lateral forces. The amount of centrifugal force is $\frac{UV^2}{32.2R}$ in pounds per linear foot applied at the top of the rails, where U = the equivalent uniformly distributed live load which will produce the same live-load bending moment used in the design, V = the velocity of the train in feet per second, and R = the radius of the curve in feet. The velocity is often specified as $60 - 2\frac{1}{2}D$ miles per hour, where D = the " degree " of curvature in degrees. By substituting an equivalent expression in feet per second for V, and the approximate value of $5730 \div D$ for R we obtain the following more convenient expression for centrifugal force = $\frac{UD(24 - D)^2}{13,700}$ in pounds per linear foot. If the length of the span is long, or the degree large, it may become necessary to take into account the eccentricity of the load on the bridge whereby one girder receives more than one-half the load. This is treated more fully in books on Bridge Design.*

* For example, Kirkham's "Structural Engineering," McGraw-Hill Book Co., Inc., New York; Waddell's "Bridge Engineering," John Wiley and Sons, Inc. New York; or Marburg's "Framed Structures and Girders," Part I, McGraw-Hill Book Co., Inc., New York.

CHAPTER XXXIV

THE THEORY AND PRACTICE OF RIVETING

SYNOPSIS: A general discussion of the construction of riveted joints and of how their strength is determined. The application of the principles involved to typical problems is shown in subsequent chapters.

1. **Rivets** are used to connect the different members of a structure to one another and also to fasten together the component parts of each member. A rivet is composed of a cylindrical shank with a head at one end. The holes in the parts to be connected are made $\frac{1}{16}''$ larger than the nominal diameter of the rivet shank so that the rivet may be inserted more easily. The length of the rivet must be greater than the thickness of the metal through which it passes, so that enough metal will protrude to form the second head. The rivet is heated, then put in position and "driven" until the second head is formed and the shank is upset to fill the enlarged hole, as explained on page 30 : 4. The parts connected are held in position by temporary bolts until rivets are driven in the remaining holes, after which the bolts are replaced by rivets. For this reason no piece should be connected by less than two rivets. Conditions sometimes justify the use of a single *bolt* but never a single rivet lest the piece become twisted during the process of riveting.

2. **Shop and Field Rivets.** — Rivets which are driven in the shop are more effective than those driven in the field. Shop rivets are usually driven by machines which develop sufficient pressure to insure complete upsetting. Field rivets must be driven by smaller machines or by hand. In the shop, the members are conveniently supported by skids, trucks, or cranes. In the field, the rivets must be driven at a disadvantage on account of their comparatively inaccessible positions in the structure. It is customary to allow a smaller unit stress for field rivets, than for shop rivets. It is therefore imperative that the draftsman know which rivets are driven in the shop and which in the field. In general, shop rivets are used wherever possible because they are not only better, but they can be driven more cheaply. Field rivets must necessarily be used for connecting to each other members which are shipped separately, but shop rivets are used for holding the component parts of each member together.

3. **Position in Member.** — Rivets are driven at right angles to the line of the stress which they are to transmit from one part to another. The number of rivets required in an ordinary connection is found by dividing the total stress by the limiting value of one rivet. This limiting value will now be considered.

4. The design of a riveted joint is based upon several **assumptions.** Some of these are disputable, but the error in one assumption often tends to compensate the error in another, so that the results obtained are quite satisfactory. The usual assumptions are that:

(*a*) the driven rivet completely fills the hole,
(*b*) the effective area of a compression member is not reduced by the rivets,
(*c*) the stress in a tension member is distributed uniformly over the net area of cross section,
(*d*) the stress is equally distributed among all the rivets of an ordinary concentric joint,
(*e*) the friction between adjacent parts is neglected,
(*f*) the bending of the rivets is ignored except for long rivets, and
(*g*) rivet heads should not be subjected to tension.

Assumptions (*a*), (*b*), and (*c*) are made in the design of tension and compression members (pages 206 : 4 and 211 : 2). Shop-driven rivets probably fill the holes because sufficient pressure is exerted to compress the rivets enough to overcome any shrinkage due to the rivets cooling. Field rivets do not so surely fill the holes but this fact is discounted in the specifications by allowing a lower unit stress. Inspectors should test each rivet with a hammer, and loose rivets should be redriven. A slight looseness probably would not impair the strength of a connection materially after the "initial slip" has taken place, unless subjected to alternate tensile and compressive stresses. That the stress is equally distributed among all the rivets (assumption *d*) is impossible as may be seen from Fig. 229 (*a*). This shows two bars fastened together by three rivets. The total stress is 18,000#. If the first rivet transmits a third of the total stress from one bar to the other, then between the first two rivets one bar would carry 6000 and the other 12,000. Since the strain is proportional to the stress (page 197:3) the distance between rivets could not be kept equal in the two plates unless the area of each bar was changed at each rivet which would be impractical. This difference in distance would at once cause unequal distribution of the stress among the rivets. It is probable, however, that the average value of the rivets in a concentric connection is constant, and the allowed unit stress may be considered an average value. Eccentric connections must be treated differently, as explained in Chapter XXXVI, page 237. The friction between the adjacent parts of a riveted joint (assumption *e*) is considerable, especially when the rivets act in double shear and are machine driven. The American Bridge Company has found from the tests on their standard beam connections (page 83 : 6) that when the web is enclosed between two connection angles, the value of each rivet is about 25% greater than when a single angle is used. This is on account of the friction. Except for these angles used under average conditions, it is common practice at present to ignore friction. The consequent increased efficiency compensates at least in part for the deficiencies due to the other assumptions. Although the effect of bending short rivets is disregarded (assumption *f*), the bending of long rivets must be considered. It is unnecessary to design them as cylindrical beams as in the case of pins (Chapter XLI, page 278). More commonly the number of rivets is increased when the grip of the rivets (i.e., the thickness of the parts connected) is more than four times the diameter. The usual increase in number is 1% for each additional sixteenth of an inch in the grip. See page 235 : 2 for the effect of loose fillers. The effect of axial tension on rivets (assumption *g*) is not well understood. Many engineers maintain that no dependence should be placed upon rivets in tension because the heads are liable to be pulled off. Others maintain that considerable tension can be resisted, and tests substantiate their claim, but the safe values to allow have not been definitely determined. It is better wherever possible to avoid connections in which rivets are subjected to axial tension. If this is impractical it is well to substitute bolts for the rivets, because they are stronger in direct tension.

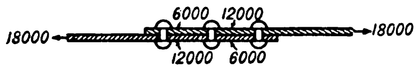


Fig. 229 (*a*).

1. **A riveted joint is designed** to resist the tendency to shear the rivets, and the tendency for the rivets to tear through one or more of the connected parts. The action of the shearing forces may be visualized by imagining a small hole bored through two overlapping steel plates and an ordinary wooden match stick driven through the hole. If one plate is made to slide along the other, the wooden match will be sheared off as if cut by shears. Similarly, if three pieces of cardboard are placed together with a wooden match passing through them, and the middle piece is slipped along the other two, remaining in contact, the match will tear through the cardboard. A riveted joint may fail, therefore, either in shear, by the shearing of the shanks of the rivets, or in bearing, by the rivets tearing through one or more of the connected parts. A tension joint may also fail in one of three other ways, 1st, in tension, by one of the parts tearing along the line of least resistance; this must be considered in the design of the main member (Chapter XXXII, page 206), and in the design of the details, as for example page 286 : 1. 2nd, by the rivets tearing out at the edges of one or more parts; this may be prevented by the use of standard edge distances (page 69 : 3) which are determined accordingly (compare page 256 : 3). 3rd, by the bending of a lap joint

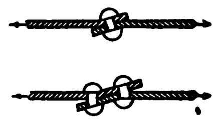


Fig. 229 (*b*).

so that the rivet heads are pulled off, as shown exaggerated in Fig. 229 (*b*); this is unusual and is not an important consideration except in tank and pipe work. The limiting value of one rivet in a joint designed for tension or for compression is determined either by the resistance in shear or in bearing, whichever is less.

1. The strength of one rivet in either shear or bearing is determined from the **nominal diameter** and not from the diameter of the driven rivet after it has been upset to fill the hole which is usually $\frac{1}{16}''$ larger. Most specifications have clauses to this effect. On account of the inaccuracies of the shop work, the holes in different connecting parts may not be in perfect alignment, and hence the upset shank will not be a true cylinder. As a result, the effective area of cross section of the rivet in the plane of contact of the connected parts may not exceed the nominal area. Since the rivet tends to shear along this plane it is important that the shearing value be limited to the strength of the rivet before upsetting. For the sake of uniformity and to provide for the incomplete upsetting of the rivet it is customary to limit the bearing value in a similar manner.

2. **The strength of a rivet in shear** depends upon the area of cross section of its shank, which is the area of a circle of a diameter equal to the nominal diameter of the rivet. The different parts connected may be so arranged that the rivets will tend to shear in either one or two planes. If all the parts which tend to move in one direction are on the *same side* of the parts which act in the opposite direction, the rivets tend to shear in one plane, as shown in Fig. 230 (*a*), and they are said to be in "**single shear.**" If the parts which act in one direction are *between* the parts which act in the opposite direction, the rivets tend to shear in two planes, as shown in Fig. 230 (*b*), and they are said to be in "**double shear.**" In ordinary practice it is unnecessary to consider more than two shearing planes. The first step in finding the limiting value of one rivet is to determine whether the rivets are in single or double shear. The unit stresses in shear on shop rivets allowed by different specifications are 10,000, 11,000, or 12,000 pounds per square inch. The corresponding values for field rivets are 8000, 9000, and 10,000. The strength of a rivet in single shear is equal to the area of a circle of a diameter equal to the nominal diameter of the rivet multiplied by the allowed unit stress per square inch. The strength of a rivet in double shear is twice that of the same rivet in single shear because the resisting area is doubled. Rivet values are tabulated at the end of the book, as explained on page 231 : 1.

Fig. 230 (*a*).

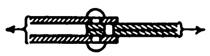


Fig. 230 (*b*).

3. **The strength of a rivet in bearing** depends upon the diameter of the rivet and the thickness of the metal in which it bears. The bearing surface is cylindrical, but it can offer no more resistance to forces parallel to the line of stress than a flat surface, if indeed as much, as shown in Fig. 230 (*c*). The effective area is therefore considered to be a rectangle, the dimensions of which are the nominal diameter of the rivet and the thickness of the metal in which it bears. If two plates of different thicknesses are riveted together, the rivets would naturally tear through the thinner plate before they would through the thicker plate, and the bearing value of each rivet would be determined by the thickness of the thinner plate. If more than two parts are connected by the same rivets, the rivets would have to tear through all the parts which act in the same direction and not merely through a single part. The bearing value of each rivet would then be determined by the thinner combined thickness of all the parts which act in either direction, and not necessarily by the thickness of the thinnest part. This is true whether the rivets are in single or double shear. Thus in Figs. 230 (*a*) and (*b*), the total thickness of the two plates acting toward the left should be compared with the total thickness of the two plates acting toward the right, and the smaller value used in determining the bearing value regardless of whether the plates which act in one direction are together as in (*a*), or separated as in (*b*). The unit stress in bearing allowed by the specifications is usually twice the corresponding unit stress in shear. The strength of a rivet in bearing is equal to the unit stress multiplied by the product of the nominal diameter of the rivet by the limiting thickness of metal explained above. Rivet values are tabulated at the end of the book, as explained on page 231 : 1.

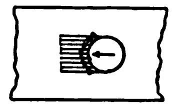


Fig. 230 (*c*).

4. **Bolts** (except specially fitted turned bolts) do not fill the holes and they are not so effective in shearing or in bearing as rivets. They are not used in important connections, but they are often used for holding

secondary members in position. The unit stress in shear is usually taken about 1000 pounds per square inch less than the unit stress for field rivets. It is sometimes specified as two-thirds the value for field rivets. The unit stress in bearing is twice the unit stress in shear.

1. To find the **number of rivets required** in an ordinary connection, first decide whether they are to be shop or field driven, and whether they are in single or double shear. Find the corresponding shearing value of one rivet of the given diameter for the specified unit stress. Next determine the limiting thickness of metal and find the corresponding bearing value. The smaller of these two values is the limiting value of one rivet, and the number of rivets required may be found by dividing the total stress by this limiting value. By means of the tables mentioned below, the number of rivets may be found directly opposite the value which equals or exceeds the given total stress. The limiting value may not be the same for all of the rivets in a given connection because some may be in single shear and the rest in double shear, or else some rivets may have a different bearing value from the rest. In this event the strength of the rivets with one limiting value must be added to that of the rivets with another limiting value to give the total strength of the connection. Tables of rivet values are given on pages 308 to 311, one page for each diameter of rivet. On each page are values for unit shearing stresses varying by thousands from 7000 to 12,000 pounds per square inch, and bearing values for unit stresses which are twice as great. Other values may be found by direct proportion. The values for unit stresses commonly specified for shop rivets are given at the left of each page, and opposite these are values for field rivets and bolts which are usually allowed by the same specifications. The values in the upper table are specified by the American Railway Engineering Association, and are in common use. In each group are given the single and double shear values and the bearing values in different thicknesses of metal not only for one rivet but for any number from one to ten. Values for more than ten rivets may be found by addition or by multiplication, as for example, the value for 14 rivets is the sum of the values for 10 and 4 rivets, is twice the value of 7 rivets, or is 14 times the value for 1 rivet. If desired, the value for one rivet may be found to one more decimal place by dividing the value for 10 rivets by 10. For convenience a dotted line is placed in the table of bearing values to show the relation of the shearing value. All bearing values to the left of the dotted lines are less than the single shear value, while all to the right are greater than for single shear and less than for double shear. The bearing values in metal thicker than the thicknesses shown are greater than for double shear. In the tables the bearing values are given only for thicknesses which are multiples of $\frac{1}{16}''$. It is usually close enough to take the web thicknesses of I-beams and channels to the nearest sixteenth from the tables on pages 322 or 323, although more exact values may be found by multiplying the web thicknesses in hundredths by the rivet diameter and by the unit stress. Except for large numbers of rivets, the latter method should not be necessary. The fractions given in the tables mentioned above should be used in preference to those given on pages 298 to 301 because in some cases they are $\frac{1}{16}''$ less, as explained at the tops of the tables. The former are used in designing, for the sake of safety, while the latter are used in drafting, for the sake of clearance.

2. **Flattened and Countersunk Rivets.** — It is often necessary to flatten rivet heads to $\frac{3}{8}''$ or $\frac{1}{4}''$ in height in order to furnish clearance for erecting other members. A flattened rivet may be considered as strong as an ordinary button-head rivet. The effective part of the rivet, i.e., the upset shank, is the same for both since only the head is altered in shape. Countersunk rivets are used instead of flattened rivets when greater clearance is required. A countersunk rivet head is shaped as shown on page 304, and the rivet hole must be reamed out or recessed to provide space for the head, much as hard wood is reamed to provide for the head of an ordinary flat-headed wood screw. The head of a countersunk rivet may not fit the reamed hole exactly, hence it may not be flush with the surface of the piece in which it is countersunk. If necessary, any projection should be chipped off with a pneumatic chisel, or otherwise, to provide a smooth bearing surface. This is done on the bottoms of bearing plates so that the whole plates will rest on the supports and not simply on the rivet heads. The heads are also chipped if another steel surface is to be placed in contact after the rivets are driven. Countersunk rivets may be used without being chipped where a slight projection is not objectionable, but where a rivet flattened to $\frac{1}{4}''$ would project too much. A countersunk rivet will not project over $\frac{1}{8}''$.

Practice is not uniform regarding the strength of countersunk rivets. Some engineers use them for holding fillers and base plates in position, but never count upon their carrying much, if any, direct stress. Other engineers count them one-half as strong as button-head rivets. If the thickness of metal in which a rivet is countersunk is no greater than the depth of the head (page 304), the entire bearing of the plate on the rivet is on a conical surface, and the sharp cutting edge of the plate tends to cut the rivet at the junction of the head and the shank. However, the bearing area is increased and the bearing on the sloping surface causes transverse components which tend to increase the friction between the parts connected. It seems as if the minimum allowance of one-half value is justified. As the thickness of the plate increases, part of the plate bears upon the rivet shank, and the value of the rivet increases. When the plate thickness equals nine-tenths of the diameter of the rivet, the bearing value on the shank alone equals the single shear value of the rivet, and there is no question but what the *full* rivet value may be counted. Rivets should be countersunk only where unavoidable because of their expense and their uncertain value.

1. **Indirect Riveting.** — The number of rivets required to transmit a given stress should be increased 50 % if loose fillers are inserted between the connected parts. This is discussed more fully on page 235 : 2. Similarly, when splice plates are not in direct contact with the parts spliced, the number of rivets should be increased $33\frac{1}{3}$ % for each intervening part.

2. **Typical riveted connections** are designed in the following chapters. On account of the inaccuracies in the usual shop work, it is not wise to combine two different types of connection. For example, a girder may be supported by seats or brackets, or else by web connection angles, but either one or the other should be designed to carry the full load. A small erection seat may be furnished to support the girder until the rivets are driven in the web connection angles, but the strength of the latter should not be reduced as a result, because it would be difficult to insure both connections acting simultaneously.

CHAPTER XXXV

RIVETS IN TYPICAL CONNECTIONS

SYNOPSIS: The principles of the preceding chapter are illustrated by the determination of the number of rivets in a few simple connections. Further application of the principles are shown in subsequent chapters.

1. In the preceding chapter were presented the principles involved in the design of riveted joints. The application of these principles will now be shown by **typical riveted connections.**

2. **Gusset Plate** — *Continuous Chord.* — This method of connecting the members of trusses, latticed girders, bracing, etc., is very common, as shown by the typical drawings of Part II. The plates of a truss are usually made of uniform thickness unless it seems desirable to increase the thickness of a few of the more important plates in order to reduce their size by reducing the number of rivets required. Gusset plates are usually from $\frac{5}{16}''$ to $\frac{5}{8}''$ thick, $\frac{5}{16}$ and $\frac{3}{8}$ being commonly used except for bridge trusses or other heavy work. Any cross section of the plate must have sufficient area to carry the maximum stress which may come on the plate at that section. This is largely a matter of judgment rather than actual design in most cases. For illustration, let us consider the plate at panel point K of the roof truss of Fig. 116. The number of rivets in the web members are determined by the corresponding stresses, provided no less than two rivets are used at the end of any member (page 228:1). These rivets usually determine the size of the plate (page 76 : 1). The number of rivets in the continuous chord is usually fixed by the practical rules for spacing, so that the edge distances will not be excessive and so that the distance between rivets will not exceed 6″. These rivets do not need to transmit the chord stress in the panel on either side of the joint, but merely the *difference* between these stresses. The continuous chord angles transmit the smaller stress from one panel to the next. The stresses (in thousands) for which this joint is designed are shown in Fig. 233. Each web member is composed of only one angle, so the rivets are in single shear. The limiting thickness of metal is $\frac{5}{16}''$, the angles and the plate being the same. From the table at the top of page 309 we find that the single shear value of a $\frac{3}{4}''$ shop rivet (5.3 thousand pounds) is less than the bearing value in a $\frac{5}{16}''$ plate (5.6) so the former must be used. By following this column in the table until a value is found which equals or exceeds the required stress, the corresponding number of rivets is determined. A single rivet would carry the smaller stress but a minimum of two would be used. Two rivets are also sufficient in the other member. The chord is composed of two angles, and the plate is inserted between them. The rivets are therefore in double shear and the limiting value is the bearing value in the $\frac{5}{16}''$ plate, the plate being thinner than the combined thickness of the angles. Two rivets would carry the increase in stress ($7 = 52 - 45$) but for practical reasons three are used.

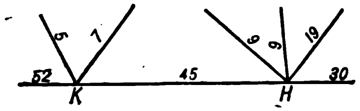


Fig. 233.

3. **Gusset Plate** — *Spliced Chord.* — Where the chord is not continuous, provision must be made to transmit the full stress in each member. A stress which is not too large may be transmitted to the gusset plate by rivets in one leg only. Whenever the number of rivets exceeds about 8, it is well to connect both legs if practical. The outstanding leg may be connected to the gusset plate by means of a small

connection angle, as for example at the left end of the chord angles of the preceding problem. The outstanding legs of two adjacent chord members may be connected by means of a splice plate which transmits part of the stress directly from one member to the other. Often a plate designed for another purpose may be utilized as a splice plate. For example, let us consider the joint at panel point *H* of the roof truss referred to in the preceding paragraph. Each member is composed of two angles, and the gusset plate is placed between them. All rivets in the gusset plate are therefore in double shear, and in value they are limited by the bearing in a $\frac{5}{16}''$ plate. From the table at the top of page 309, two shop rivets are found sufficient for the stresses in each of the two left-hand web members (Fig. 116), but four are required in the right-hand member. The two chord members are so arranged that the outstanding legs may be connected by means of a splice plate. The proportion of the stress to be taken by the rivets in the different legs depends upon the special requirements of each problem. It may be desirable to proportion the rivets approximately to the relative areas of the angle legs. There must be at least enough rivets in the gusset plate to prevent the web members from moving the plate along the chord. This tendency is measured by the difference in the chord stresses, as in the preceding problem. Unlike the gusset plate, the splice plate serves simply to transmit stress from one chord member to the other and nothing is gained in making one-half the splice stronger than the other half. If shop rivets are used in one-half and field rivets in the other, or if different numbers of rivets are used, the stress cannot exceed the strength of the weaker half. This should be borne in mind when a plate used for another purpose serves as a splice plate. In this problem, for example, a plate in the system of bottom chord bracing (Fig. 140) is so used, but the holes are not symmetrical about the joint. Only four rivets can be counted upon to transmit the chord stress, because there are only four in the right-hand member. From the table, four field rivets in single shear are worth 18 thousand pounds. The number of *shop* rivets bearing in the $\frac{5}{16}''$ gusset plate required to carry the remaining stress of $27 = 45 - 18$ in the left-hand chord member is 5. Similarly, the number of *field* rivets required in the right-hand member to carry $12 = 30 - 18$ is 3

1. **Symmetrical Gusset Plate.** — In order to make connecting par alike, or symmetrical about a reference line, the same number of rive must be used in both parts even though some of the rivets are drive in the shop and the rest in the field. For example only one-half of symmetrical roof truss is drawn, and both halves of the truss are mad from the drawing, whether the truss is fully riveted in the shop or shippe in sections. In the latter case the central peak plate (panel point *I* Fig. 116) is usually riveted in the shop to one section in order to reduc the number of field rivets. Since it is impractical to make the plate un symmetrical, the number of rivets must be determined by the value c the *field* rivets.

2. **Web Connection for Beams.** — Standard connection angles ar often used for supporting I-beams and channels under average cond tions of loading. Their use not only simplifies the drafting and the sho work but reduces the number of connections to be designed. Thes angles must not be used blindly for all conditions, such as for very shor spans or for heavy concentrated loads. Draftsmen soon become familia with the limitations of standard connection angles. All doubtful case should be investigated, and special connections should be designed wher ever necessary. Four-inch legs are used as a rule when the rivets ma be put in a single line, and six-inch legs for double lines. Thicknesse of $\frac{3}{8}''$, $\frac{7}{16}''$ and $\frac{1}{2}''$ are used. The lengths depend upon the number c rivets. Angles are regularly used in pairs except in crowded places wher single angles may be used. The shop rivets which connect the angle to the supported beam are accordingly in double or single shear. The must transmit the maximum end reaction.* The field rivets which con nect the angles to the supporting member must transmit the same maxi mum reaction, acting in single shear. If another beam is connected t the web of the supporting member by means of the same field rivets the rivets must also transmit the combined reactions of the beams, actin in double shear. Enough rivets must be used to satisfy either of thes

* These rivets are sometimes figured according to the method of eccentric con nections (next chapter), but ordinarily this is not done because it is considered tha the tension in the rivets through the outstanding legs will restrain the connectio angles sufficiently to overcome the effects of eccentricity. In fact the tendency is t use fewer rivets rather than more on account of the friction (page 228 : 4).

requirements. The maximum number will usually be required when both beams are loaded, but not always. (Compare page 235: 2) For example, let us find the number of $\frac{3}{4}''$ rivets required in the two angles connecting a 15″ I 42# to a $\frac{5}{16}''$ girder web. The beam is 10′ 0″ long, and supports a total load of 6000#/ft., including its own weight. The rivet values given in the bottom table of page 309 are specified. The maximum reaction is 30,000# = 6000 × 5. The shop rivets are in double shear, being limited by the bearing value in the $\frac{3}{8}''$ web (page 322) of the I-beam. The number of shop rivets required is found from the table to be 6. The number of field rivets in the outstanding legs is determined by the single shear value to be 9, but 5 would be used in each angle to make them alike. If another beam of the same size and with the same load is to be connected to the opposite side of the girder web by the same field rivets, the rivets must be proportioned for the combined load of 60,000#. In this case the rivets would be in double shear and limited by the bearing in the $\frac{5}{16}''$ web. The number required would thus be increased from 9 to 16.

1. **Seat Connection for Beams.** — A seat angle is used to support the lighter beams of office buildings, as shown in Fig. 89 (*a*), (*b*). The beams are held in place by small top or side angles, but the seat angles are designed to take the whole load. The thickness of the seat angle is increased with the load in order to prevent bending, $\frac{1}{2}''$ being used for loads up to 12,000#, $\frac{5}{8}''$ up to 15,000#, and $\frac{3}{4}''$ up to 18,000#.* For heavier loads, web connections are preferred unless two beams have to be supported by the same rivets, in which event seat angles with stiffeners are chosen on account of erection (page 89 : 2). One or two stiffening angles may be required, the outstanding legs being designed so that the portion outside the fillet of the seat angle will carry the load in bearing at 24,000#/sq. in. (page 267 : 1). The number of rivets in the stiffeners is determined in the usual manner. Examples of this type of connection are shown in Figs. 104 and 133. When two stiffeners are used it is not necessary to rivet their outstanding legs together unless exposed to the weather. When the gage in a single stiffening angle exceeds $2\frac{1}{2}''$ or 3″, the connection should be tested for eccentricity (page 237 : 3). For example, let us find the number of $\frac{3}{4}''$ rivets required to connect a seat angle to a $\frac{7}{16}''$ girder web to support the end of a 12″ I $31\frac{1}{2}$#, the reaction being 12,000#. From the second table on page 309 we find that 3 shop rivets are required. If a similar beam seat must be provided on the opposite side of the web, the combined load will be 24,000# and the rivets will be in double shear. Using the bearing value in the web we find that 4 rivets are required.

* Standards of the American Bridge Company.

2. **Stringer Connection.** — The connection of a stringer to the floor beam of a railroad bridge may be illustrated by the use of the data given on page 225 : 1. A similar stringer is detailed in Fig. 98, and the corresponding floor beam in Fig. 99. The maximum end shear due to live load is 60,000#, found by placing wheel 2 at the extreme end of the 15-ft stringer. This must be increased by $\frac{300}{315}$ per cent for impact, and by the dead load of the track and the stringer, thus: $119{,}900\# = 60{,}000 \times \frac{615}{315} + (225 + 150)7.5 =$ the maximum total end shear. The number of shop rivets which connect the angles to the stringer web is determined from this maximum shear. The field rivets which connect the angles to the floor-beam web must resist either this maximum end shear (acting in single shear), or else the maximum load from two stringers (acting in double shear). Either condition may give the larger number of rivets so both must be considered. This is because the combined load or floor-beam reaction for moving concentrated loads can never be twice the maximum end shear. The maximum floor-beam reaction due to the live load was found on page 194 : 2 to be 82,000#. Since this is obtained when two panels are loaded, the impact percentage is $\frac{300}{330}$. The total floor-beam reaction is therefore $162{,}200\# = 82{,}000 \times \frac{630}{330} + (225 + 150)15.$ Using the first table on page 310, the number of $\frac{7}{8}''$ shop rivets bearing in the $\frac{3}{8}''$ stringer web required to carry 119,900# is 10. Rivets which pass through fillers are less effective than those which connect the parts directly without intervening fillers. The common clause in the specifications requires that the number of rivets should be increased 50% if they pass through fillers. These extra rivets may be used to connect the fillers to the web without passing through the angles; in fact it is

preferable to so place them wherever practical. The total number of rivets required to connect the angles and fillers to the stringer web is therefore 15, at least 10 of which should pass through the angles. The rivets which pass through the flange angles should not be counted because they will be fully developed in transmitting the flange stress. The number of field rivets in single shear required to carry 119,900# is 20 while the number bearing in a $\frac{7}{16}$" floor-beam web required to carry 162,200# is 22. If the relative depths of stringer and floor beam necessitates the use of fillers between the connection angles and the floor-beam web, as in Fig. 99, the number would be increased 50% to 33. Most of the extra rivets may be placed outside of the angles or else be countersunk beneath them, thus holding the fillers in position and minimizing the number of field rivets. When shop rivets are thus substituted for field rivets a corresponding reduction in number may be made.

1. **Floor-beam Connection.** — In the alternate form of connection shown in Fig. 99, the floor beam is connected to the stiffening angles of the girder by means of a gusset plate. If this plate is placed betwee two stiffening angles, the rivets are in double shear and usually limite by the bearing value in the plate. If only one angle is used the rivet are in single shear. The total reaction at the end of the floor beam c the preceding problem would be equal to one-half the total load, or th load at one stringer point plus one-half the weight of the floor beam, thus 163,500# = 162,200 + 175 × 7.25.

The discrepancy between this value and the panel load on the girder designe on page 225 : 1 is due to the different loaded lengths which determine the impac percentages.

2. **Other Connections.** — The rivets in eccentric connections ar discussed in the following chapter; those in the flanges of plate girder in Chapter XXXVII, page 241; in the cover plates of plate girders Chapter XXXVIII, page 259; in stiffening angles, Chapter XXXIX, pag 266; in splices, Chapter XL, page 270; in reinforcing plates, Chapte XLII, page 284; and in column bases, Chapter XLIII, page 288.

CHAPTER XXXVI

RIVETS IN ECCENTRIC CONNECTIONS

SYNOPSIS: The determination of the number of rivets required to resist the tendency of a connection to rotate, as well as to resist the direct stress.

1. **Definition.** — Whenever practicable the rivets in a connection should be arranged symmetrically about the line of action of the resultant force so that the rivets on one side balance those on the other and overcome any tendency of the connection to twist. The rivets may then be considered to resist the direct shearing forces only, as in the connections of the preceding chapter. If the rivets are unsymmetrically placed, particularly if all the rivets are on one side of the force or forces to be resisted, the connection is said to be "eccentric," and special attention must be given to its design. The tendency of an eccentric connection to rotate is often so great that the stresses on the rivets due to moment are larger than those due to the direct load.

2. The commonly accepted **method of designing** an eccentric connection is based upon assumptions which have proved satisfactory under usual conditions even though they may not be applicable to extreme cases. Formulas, tables, and diagrams have been devised to facilitate the design, but only in special cases * have they proved distinctly more satisfactory than the more general method of finding the number of rivets by trial. The number and the spacing of the rivets in an eccentric connection are assumed and then the efficiency of the whole connection is investigated. The number depends not only upon the load and the limiting value of each rivet, but also upon the spacing of the rivets and the amount of the eccentricity. No rule can be given to aid the designer in making his first assumption, but each trial serves as a guide for subsequent trials. Naturally, the number first assumed should be greater than the number which would be required to carry simply the load if the connection were concentric. The number of trials required depends largely upon the experience and the judgment of the designer.

3. The application of the method of investigating the strength of an eccentric connection by means of a working rule is comparatively simple, but the underlying **theory** is more complex. A typical connection is shown in Fig. 237, in which O is the center of gravity of the group of rivets, P is the resultant force which tends to cause rotation of the plate, and e is the eccentricity or the perpendicular distance from O to the line of action of P. The rivets are not all equally effective in resisting rotation, for those that are farthest from the center of rotation are worth most while those near this center are worth comparatively little. The stresses in the rivets are proportional to the distances from the rivets to the center of rotation, and they act in lines normal to lines drawn from the rivets to this center. If the rivets resisted simply the rotation, the center of rotation would be at O, the lines of action of typical rivets being

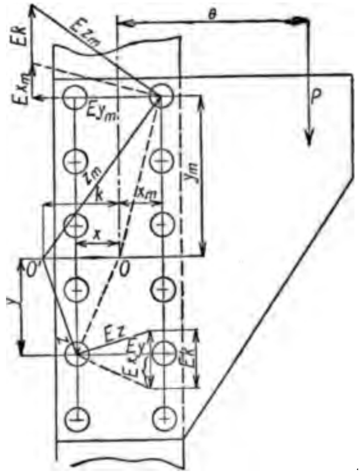


Fig. 237.

* See diagrams of F. W. Seidensticker in *Engineering News-Record* of June 19, 1919.

represented by the dashed lines in the figure. Usually, however, each rivet must also support its share of the direct load. The effect of this is to increase the vertical component of the stress in each rivet by a constant amount (Ek) and to change the center of rotation to O' on a line drawn through O perpendicular to the line of action of the force P. The combined stress (Ez) on any rivet acts in a line normal to a line drawn from the rivet to the center of rotation O'. The rivet which is farthest from the center of rotation O' has the largest combined stress (Ez_m) and it is therefore the critical rivet in determining the strength of the connection, inasmuch as its stress must not exceed the limiting value of a rivet under the given conditions (page 231 : 1). In order to derive the expressions used as a basis for a working rule, let us refer our *rectangular* coördinates to the center of gravity of the group of rivets, and our *polar* coördinates to the center of rotation.

O = the center of gravity of the group of rivets
O' = the center of rotation
P = the resultant force to be resisted
e = the eccentricity, or the perpendicular distance from O to P
k = the distance from O to O'
x = the horizontal distance from O to any rivet
y = the vertical distance from O to any rivet
z = the direct distance from O' (not O) to any rivet
z_m = the maximum value of z
x_m and y_m = the values of x and y which correspond to z_m
E = the resultant stress on a rivet at unit distance from O'
N = the number of rivets in the whole connection.

Since the stress in any rivet is proportional to the distance from the rivet to the center of rotation O', the resultant stress in a rivet at distance z is Ez, and its moment about O' is Ez^2. From the figure, $z^2 = (k \pm x)^2 + y^2$. By similar triangles, since E is constant, the *horizontal* component of Ez is Ey, and the *vertical* component is composed of two parts added algebraically, viz.: Ex due to rotation, and Ek due to the direct load. Since E and k are both constant, Ek is the same for all rivets and $Ek = \frac{P}{N}$; also $\Sigma Ek = P$, and $\Sigma Ek^2 = Pk$. In order to satisfy the moment equation of equilibrium, the total resisting moment of the rivets must equal the moment of the force P about O', or $\Sigma Ez^2 = \Sigma E\left[(k \pm x)^2 + y^2\right] = P(k+e)$, whence

$$\Sigma Ek^2 \pm \Sigma 2Ekx + \Sigma Ex^2 + \Sigma Ey^2 = Pk + Pe.$$

Since O is at the center of gravity of the rivets, $\Sigma x = 0$, hence $\Sigma 2Ekx = 0$. Substituting this value and $\Sigma Ek^2 = Pk$, we have

$$\Sigma Ex^2 + \Sigma Ey^2 = Pe, \qquad \text{or} \qquad E = \frac{Pe}{\Sigma x^2 + \Sigma y^2}.$$

The maximum resultant stress (Ez_m) will occur in the rivet which is at the maximum distance (z_m) from the center of rotation (O'). For this rivet the Ek and the Ex_m of the vertical component are combined because they have the same sign. From the figure, $(Ez_m)^2 = (Ek + Ex_m)^2 + (Ey_m)^2$, from which the maximum stress on the critical rivet

$$Ez_m = \sqrt{\left(\frac{P}{N} + Ex_m\right)^2 + (Ey_m)^2}.$$

This maximum stress must not exceed the limiting value of one rivet as determined in the usual manner.

1. The method derived in the preceding paragraph for investigating the strength of the rivets in an eccentric connection may be summarized in the form of a more convenient working rule as follows:

> Assume the number and the spacing of the rivets and find the center of gravity (O) of the whole group. Find E (or an expression for E) by dividing the moment (Pe) of the resultant force P about O by the sum of the squares of the horizontal and the vertical distances ($\Sigma x^2 + \Sigma y^2$) from O to each rivet. Select the rivet which is at the maximum distance (z_m) from the center of rotation (O') as the critical rivet. The product of E and the *vertical* distance (y_m) from O to the critical rivet is the horizontal component (Ey_m) of the maximum stress (Ez_m). The vertical component $\left(\frac{P}{N} + Ex_m\right)$ is found by adding the direct load on each rivet $\left(\frac{P}{N}\right)$ to the product of E and the *horizontal* distance (x_m) from O to the critical rivet. The resultant

of these two components is the maximum stress on the critical rivet (Ez_m), and this should not exceed the limiting value allowed on a single rivet.

The spacing of the rivets is often predetermined by existing conditions. The number of rivets required is smaller when they are spaced farther apart, but it is impractical to exceed certain limits lest the resulting plate appear too large. The position of the center of gravity can usually be found by inspection. The sum of the squares may be found most readily by combining rivets which have equal values of x or of y, and by using a table of squares or a slide rule. The resultant stress (Ez_m) may be found graphically or by means of the diagram on page 312 quite as conveniently as with the table of squares. The limiting value of one rivet depends upon the unit stress, the diameter, and whether the rivets are in single or double shear, as in the preceding chapters. See page 231 : 1. In case the connection proves not strong enough, rivets may be added or the spacing may be increased, whichever seems better adapted to the specific problem. Often two or more rivets must be added to preserve the symmetry even though one would give the desired increase in strength. In like manner the connection may prove to be so strong that the number or the spacing of the rivets may be reduced.

1. The application of the foregoing rule is illustrated by the following **typical problems.** When the moment due to the eccentricity is resisted by more than one group of rivets, each group is designed for its portion, as illustrated in the design of the web splices of plate girders on pages 271 : 1 and 272 : 2.

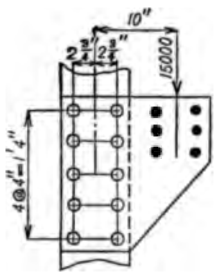


Fig. 239 (a).

First Problem. — Find the number of ¾″ shop rivets in single shear at 10,000#/sq. in., spaced 4″ c. to c. in two rows 5½″ apart, which are required to support an eccentric load of 15,000#, acting 10″ from the center of the group of rivets. The value of one rivet is 4420 and four rivets would be required to take the direct load. On account of the comparatively large eccentricity, we will assume 10 rivets as shown in Fig. 239 (*a*). The center of gravity is midway between the two central rivets. The value of x is 2¾″ for each rivet, and the value of y for the center rivets is 0, for the four outer rivets is 8″, and for the four remaining rivets is 4″. The sum of the squares is

$$396 = 10 \times 2\tfrac{3}{4}^2 + 4(4^2 + 8^2), \qquad \text{and} \qquad E = \frac{15{,}000 \times 10}{396}.$$

The value of E need not be recorded if a slide rule is used, for without changing the slide, the value of E may be multiplied by $y_m = 8$, and $x_m = 2\tfrac{3}{4}$ to give 3030 and 1040 respectively. The former is the horizontal component of the stress on the extreme rivet and the vertical component is $2540 = \dfrac{15{,}000}{10} + 1040$. The resultant of these two is 3950, which is considerably under the allowed value of 4420. If we remove two rivets, the strength of the eight remaining ones would be found as follows:

$$220 = 8 \times 2\tfrac{3}{4}^2 + 4(2^2 + 6^2) \qquad \text{and} \qquad E = \frac{15{,}000 \times 10}{220}.$$

$E \times 6 = 4090$, and $E \times 2\tfrac{3}{4} = 1880$. The resultant of 4090 and 3750 $= \dfrac{15{,}000}{8} + 1880$ is 5550 which is much too large. Often the magnitude of one or both components is such that it is unnecessary to find the resultant before discarding the trial. In this case 10 rivets would be used because 4″ spacing is specified and it would be impracticable to use 9 rivets even though they proved strong enough. If the spacing were not fixed it might be either reduced so that a smaller plate could be used with 10 rivets, or else increased so that 8 rivets would suffice.

Second Problem. — Find the number of ¾″ rivets required at 12,000#/sq. in. to connect the ⅜″ plate to the two vertical angles of the bracket shown in Fig. 239 (*b*). The rivets are in double shear, and the limiting value of each rivet is 6750, the bearing value in a ⅜″ plate. Let us assume six rivets spaced 3″ apart as shown. The sum of the squares is $158 = 2(1\tfrac{1}{2}^2 + 4\tfrac{1}{2}^2 + 7\tfrac{1}{2}^2)$, since $x = 0$. $E = \dfrac{15{,}000 \times 8}{158}$ which multiplied by 7½ gives 5710 for the horizontal component. The

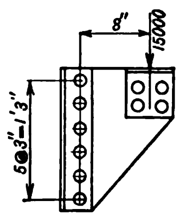


Fig. 239 (*b*).

vertical component is simply $2500 = \frac{15{,}000}{6}$ because $Ex = 0$. The resultant is 6230, which is slightly under the allowed 6750. One rivet cannot be omitted because the horizontal component alone would be 8000 $= \frac{15{,}000 \times 8}{2(3^2 + 6^2)} \times 6$. The six rivets may be placed closer together and still meet the requirements. Let us try $2\frac{3}{4}''$ spacing. The sum of the square $= 132 = 2(1\frac{3}{8}^2 + 4\frac{1}{8}^2 + 6\frac{7}{8}^2)$. The horizontal component $= 6260 = \frac{15{,}000 \times}{132}$ $\times\, 6\frac{7}{8}$, which combined with the vertical component of 2500 gives resultant of 6740. This solution meets the requirements almos exactly.

CHAPTER XXXVII

RIVETS IN THE FLANGES OF PLATE GIRDERS

SYNOPSIS: The rivets which fasten the flange angles to the web plate of a girder are not spaced uniformly but are proportioned according to the variable flange stress. The methods of finding the rivet pitches under different conditions are fully treated in this chapter.

1. The term "**flange rivets**" is usually interpreted to mean the rivets by which the flange angles of a plate girder are attached to the web plate, and it is so used here. Technically, however, the term might also apply to the rivets which fasten the cover plates to the flange angles, the spacing of which is discussed in the following chapter, page 263 : 3.

2. The term "**rivet pitch**" commonly refers to the longitudinal spacing of the rivets which hold the component parts of a member together, and not of those which connect one member to another, although it is sometimes used synonymously with the more general term "rivet spacing." The pitch of the flange rivets in a plate girder is the distance from center to center of adjacent rivets measured *parallel* to the longitudinal axis of the girder. If the rivets in one flange are placed on two lines they are "staggered" or alternated, but the pitch is the distance from the center of a rivet on one line to the center of the next rivet on the other line measured *not* directly, but parallel to the axis of the girder, as shown in Fig. 241.

3. Since the great majority of girders are horizontal, it is convenient to speak of "horizontal" and "vertical" stresses in the flanges for the sake of brevity. This chapter may be made to apply to vertical or **inclined girders** by substituting for "horizontal" the more general expression "parallel to the longitudinal axis of the girder," and for "vertical" the expression "perpendicular to the longitudinal axis of the girder."

4. There is a demand for a more **complete treatise** on the subject of flange riveting than is found in most books on the market. In this chapter are given the formulas and the methods used under different conditions, and also the proofs of the formulas and the reasons for making the different assumptions. At the end of the chapter the different cases

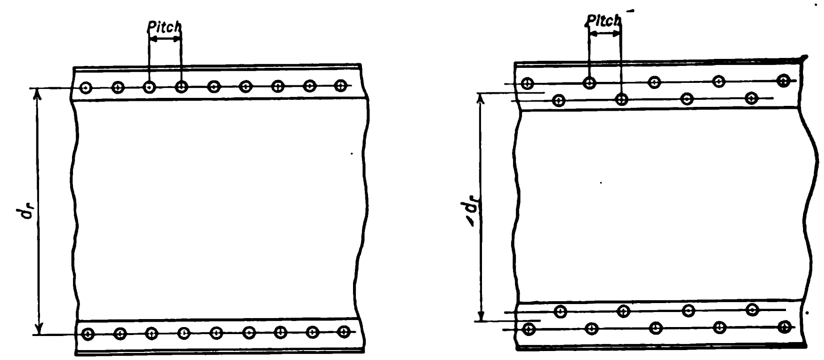


Fig. 241.

are summarized for convenient reference, but it is recommended that the summary be used only after a full understanding of the underlying theory has been gained.

5. **General Discussion.** — The flange rivets in a girder are usually spaced closer together near the ends than near the center because the

rate of increase in the flange stress near the ends is greater. If a girder is symmetrical only one-half need be considered. The maximum pitches at the ends and at other points are determined for each girder by the methods explained in this chapter. These maximum pitches are based upon the proper number of rivets required to transmit the flange stresses. The spacing must also conform to the general rules for rivet spacing given in Chapter XIII, page 68, particularly regarding the maximum and minimum values. The minimum rivet pitch should not be less than the proper value selected from the table on page 306. These values are determined by the strength of the web plate between rivets, as explained on page 255 : 2. This is an important consideration which is too often overlooked by designers. The pitch in the end panel of a girder is first determined, because if it is found to be less than the minimum allowed, some change must be made in the design of the flange, as for example: (*a*) if the vertical legs of the angles are less than 6″, they may be increased to permit the staggering of the rivets; (*b*) the value of each rivet may be increased by increasing either the diameter of the rivet or the thickness of the web plate; (*c*) vertical flange plates may be inserted between the angles and the web; or (*d*) as a last resort the depth of the girder may be increased or the length of span may be decreased. A constant pitch is used throughout a given panel as explained later. The pitch in each panel should extend at least as far as the beginning of the next panel. In spacing the rivets it may be desirable to extend one pitch beyond its panel; the next pitch need not then extend a full panel provided it extends as far from the end of the girder as the point at which the next pitch is calculated. Instead of changing the pitch at the center of a concentrated load the smaller pitch should be extended past the entire connection. Rivets adjacent to stiffening angles should be placed far enough away to give driving clearance, even though the calculated pitch is exceeded. It is unnecessary to calculate the pitch beyond the panel where it reaches the specified maximum because this maximum pitch must be used from this panel to the center. This maximum pitch never exceeds 6″, and it is usually limited to 4″ or 4½″ in crane runway girders and railroad girders or stringers when the loads rest directly upon the flanges.

1. Some of the **controlling factors** which determine the pitch of flange rivets are:

(*a*) the form of loading (fixed or moving, concentrated or uniformly distributed),
(*b*) the application of the loads (to the web or to the flange),
(*c*) the method of design adopted (whether the resisting moment of the web plate is considered),
(*d*) the depth of the girder between rivet lines,
(*e*) the limiting value of each rivet,
(*f*) the general rules which govern rivet spacing
(*g*) the strength of the web between rivets.

2. **Forces Considered.** — The web plate of a girder is designed to resist all shearing stresses, and corresponds to the web members of a parallel-chord truss. It transfers vertical forces diagonally to the flanges which are designed for horizontal stresses like the chords of a parallel chord truss. All vertical loads must be transmitted either directly or indirectly to the web plate. Fixed concentrated loads are either connected directly to the web or else transmitted to it by means of stiffening angles. Uniformly distributed loads and moving concentrated loads are either applied directly to the web through some type of solid-floor construction, or else rest upon one of the flanges (usually the top). The flange rivets must be designed to transmit to the flange angles and cover plates the horizontal stresses due to bending, and in addition they must transmit to the web plate any vertical loads which rest on the flanges. The rivets in any panel must transmit the maximum increase in horizontal flange stress in that panel. It so happens that this increase in horizontal flange stress is equivalent to the vertical shear for a certain section, and it is convenient to make use of this fact. No one should receive the false impression, however, that the rivets are proportioned for vertical forces (except in part for flange loads). Moving loads should be placed to give the maximum shear.

3. In this chapter the **depth** used in proportioning the flange rivets of a girder is the perpendicular distance between the rivet lines, and *not* between the centers of gravity of the flanges. When two lines are used in each flange, the mean depth is taken, as shown in Fig. 241. It is logical to use the depth between rivet lines because it is the distance between the lines of action of the stresses for which the rivets are designed.

The flange stresses can be transmitted from the web to the angles only by means of the rivets acting along these lines of action. The resisting moment of the stresses in the rivets is the moment of a couple, equal to the product of the stress in the rivets in either flange by the perpendicular distance between the lines of action. This resisting moment must be equal to the resisting moment of the flanges and hence to the corresponding bending moment. If the depth between rivet lines is less than the depth used in the design of the flanges, a correspondingly greater flange stress must be resisted by the rivets than by the angles and cover plates. This is analogous to the common method of designing the web splice plates which transmit the portion of the bending stresses resisted by the web plate, explained on page 272 : 2. It is also consistent with the design of pin-connected trusses in which the depth is measured between the centers of the pins and not necessarily between the centers of gravity of the chords. The increased stress which results from the use of a reduced depth may provide in some measure for the secondary stresses which exist in almost every girder due to the fact that the rivets are not placed at the centers of gravity of the flanges. The mean depth between rivet lines in inches is denoted by d_r, while in feet it becomes D_r; it should usually be taken to the nearest quarter of an inch. Advocates of the use of other depths may adapt this chapter to their needs by simply substituting another depth for d_r.

1. The different conditions under which flange rivets are proportioned are considered under eight **different cases.** For the sake of simplicity in each of the first five cases it is assumed that the girder is designed according to Case A (page 221 : 2), in which the flanges resist the whole bending moment and the resisting moment of the web plate is neglected. Case VI shows how the first five cases may be modified when the girder is designed according to Case B (page 223 : 3), in which the resisting moment of the web is considered. Cases VII and VIII show the application of the principles of the foregoing cases to special girders.

The subdivisions are as follows:

CASE I — CONCENTRATED STATIC LOADS APPLIED TO THE WEB

2. **How Applied.** — Concentrated loads are applied to plate girders only in conjunction with uniformly distributed loads. These uniform loads may be due simply to the weights of the girders themselves, or they may include other dead or live loads. For the purpose of explanation, it seems best to begin with the discussion of the separate systems of loading and to combine them later. Concentrated loads may be applied to one of the flanges as explained under Case V, but under this Case I are considered only concentrated static loads which are connected directly to the web plate by means of connection angles or stiffening angles. Concentrated static loads may be fixed in position but variable in magnitude; the rivet pitch should be determined from the maximum values.

3. **Bending Moment Theory.** — The curve of bending moments for a system of static concentrated loads is a series of straight lines, as shown in Fig. 189. The bending moment increases *constantly* from either end of a girder to the point of application of the nearest load, and also from one load to another. The rivet pitch should therefore remain constant from one load (or reaction) to the next load. There should be enough rivets between two loads to satisfy the *difference* in flange stress between their points of application. Thus there will be enough rivets between the end of a simple girder where the bending moment is zero, and any

other point to carry the flange stress due to the total bending moment at that point. It was shown on page 181 : 1 that the increase in bending moment between the points of application of fixed concentrated loads is equal to the product of the shear for a section between the loads multiplied by the distance between the loads. If we represent the vertical shear in any panel by V, the increase in bending moment in any panel of length B feet is VB, and the corresponding increase in horizontal flange stress to be taken by the rivets in the panel is $\frac{VB}{D_r}$ (page 242 : 3). If the limiting value of one rivet under the given conditions is r (usually the bearing value in the web plate, page 230 : 3), the number of rivets required in the panel is $\frac{VB}{D_r} \div r$. These rivets are equally spaced throughout the panel B, hence the pitch in feet measured horizontally from center to center of rivets is $B \div \frac{VB}{rD_r}$ or $\frac{rD_r}{V}$. Since the pitch is always less than one foot it should be expressed in inches, and the above value should be multiplied by 12, which changes D_r to d_r. Thus the pitch in inches is

$$p = \frac{rd_r}{V}.$$

1. **Shear Theory.** — The expression found in the preceding paragraph may be derived also by an entirely different method. The intensity of horizontal shear at any point of a girder with fixed loads is $v = \frac{Vq}{Ib}$ (page 202 : 1). Assuming the flange stresses to be concentrated at the rivet lines according to page 242 : 3, and neglecting the resisting moment of the web plate according to Case A (page 221 : 2), $b = t =$ the web thickness, $q = \frac{ad_r}{2}$, and $I = 2I' + 2a\left(\frac{d_r}{2}\right)^2$, where $a =$ the area of angles and cover plates of one flange, and $I' =$ the corresponding moment of inertia about an axis through the rivet line. Then $v = \frac{V}{\left(\frac{4I'}{ad_r} + d_r\right)t}$. The term $\frac{4I'}{ad_r}$ can with safety be neglected; it is usually a small fraction, particularly at the ends of the girder where its effect would be most pronounced. The intensity of horizontal shear then becomes $v = \frac{V}{d_r t}$, and the corresponding stress per linear inch is found by multiplying by the web thickness t to be $\frac{V}{d_r}$. Since the value of each rivet is r, the distance between rivets should be $r \div \frac{V}{d_r}$, or $p = \frac{rd_r}{V}$, as before.

CASE II — UNIFORMLY DISTRIBUTED STATIC LOADS APPLIED TO THE WEB

2. **How Applied.** — The weight of a girder is considered to be uniformly distributed throughout its length. The superimposed load may be concentrated, uniformly distributed, or part concentrated and part uniform. The superimposed loads may be moving loads, as discussed under Case IV (page 247 : 1), and they may be applied to the flange, as discussed under Case V (page 248 : 2). Under this Case II are considered only static uniformly distributed loads which are applied to the web. The weight of the girder may be included.

3. **Theory.** — The curve of bending moments for loads which are uniformly distributed the full length of a girder is a parabola. Since the flange stress is directly proportional to the bending moment (since $F = M_B \div D_r$), a parabola may be drawn the ordinates of which represent the flange stresses at different points, as shown by the curved line (1) in Fig. 245 (a). The flange stress is zero at the ends of a simple girder, and maximum at the center. The rate of increase is not constant, but is greater near the ends, and the rivet pitch should accordingly be smaller at the ends and larger at the center. Theoretically, no two intervals should be alike, but this is impractical because of the amount of calculation involved in determining the pitches, and because of the extra work required in spacing the rivets both in the drafting room and in the shop. It is customary to subdivide the girders into panels, to determine the pitch at the beginning of each panel, and to use this constant pitch throughout that panel, even though more rivets are thus used than are theoretically required. The panel lengths are arbitrarily chosen approximately equal to the depth of the girder, preferably equal to the depth between rivet lines. It is not necessary that the girder length be an exac

multiple of the panel length chosen, the balance being placed at the center where the same pitch usually extends for more than one panel. When intermediate stiffening angles are used, the panel lengths may be made to conform to the spacing of the stiffeners, although if the distance between stiffeners greatly exceeds the girder depth an excessive number of rivets will result. As in the case of concentrated loads it is convenient to make use of the relation between bending moment and vertical shear, but for uniformly distributed loads the shear is not constant throughout any panel. It may be easily proved that the increase in bending moment per panel for fixed loads is equal to the product of the panel length and the *average* vertical shear, i.e., the shear for a section in the middle of the panel. The corresponding increase in horizontal flange stress is equal to this product divided by the depth D_r. In Fig. 245 is shown the relation between the horizontal flange stress and the vertical shear. The ordinates in (*a*) represent the total horizontal flange stress while those in (*b*) represent the total vertical shear. The parabola (1) shows the *actual* flange stress due to the full load, and the straight line (5) shows the *actual* shear. If the increase were constant in any one panel, the stress diagram would assume the position of the dotted line (2). The corresponding shear diagram, showing the constant average shear per panel, is represented by the stepped dotted line (6). If the rivets in each panel were porportioned from this constant increase, it is apparent from either diagram that there would be a deficiency in the first half of the panel, although this would be made up in the second half so that the total number of rivets in the panel would be sufficient. In order to satisfy the increase in flange stress at the beginning of the panel, however, the rivet pitch must be determined from the shear for a section at the *beginning* of the panel instead of at the middle. In other words, if a constant pitch is used it must not exceed the smallest pitch which would be used at the beginning of the panel if the pitch were varied according to the actual stress curve. The effect of this is shown graphically by the heavy lines (7) and (3) which lie entirely outside of the theoretical lines (5) and (1). The excess in the number of rivets when spaced uniformly in this manner instead of according to the ordinates of the parabola is justified by the practical advantages.* In view of this excess,

* This excess cannot exceed $\frac{UB}{2r}$ in any panel.

each rivet pitch may be taken to the *nearest* $\frac{1}{4}''$. The method of solution is much the same as for concentrated loads, and the expression $p = \frac{rd_r}{V}$

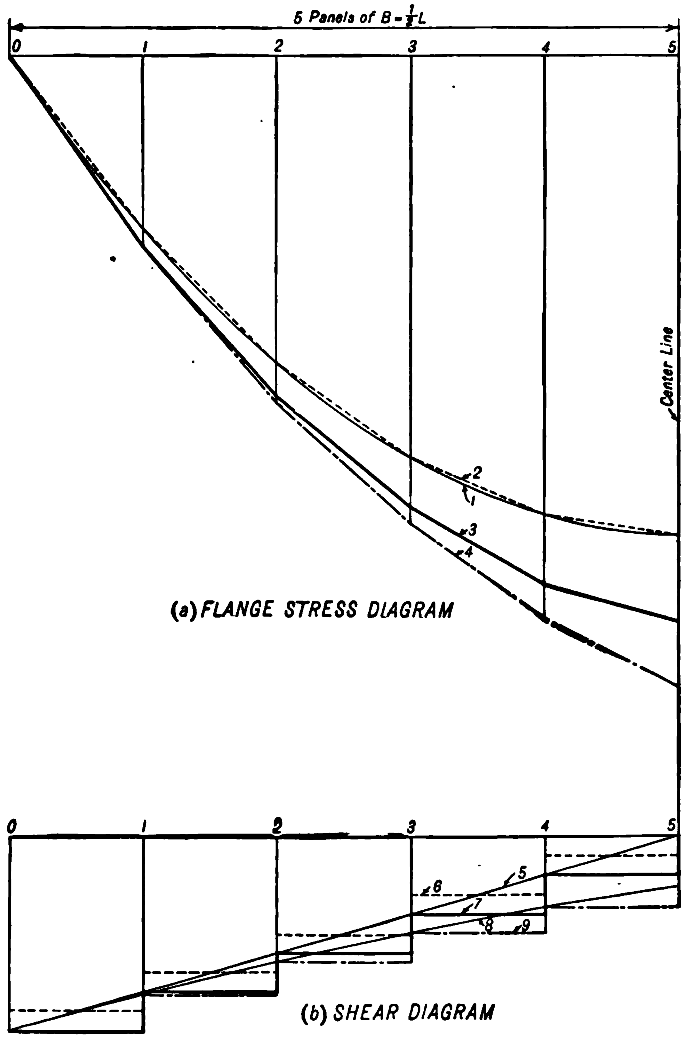


Fig. 245.

may be derived in the same manner. In the case of uniformly distributed loads the V is limited to the shear for a section at the beginning of the panel. If the panels are of equal length, the shears for different sections may be easily found by means of a constant difference. The application of this Case is shown by the following illustrative problem.

1. **Illustrative Problem** — *Uniformly Distributed Static Loads.* — Find the rivet pitches in the 18-ft. girder designed on page 222 : 1, assuming that the total load is 6500#/ft., including the weight of the girder. Use a unit stress in bearing of 24,000#/sq. in.

6750# = r = value of one ¾″ rivet bearing in a ⅜″ plate (page 309)
19.5″ = 24½ − 2 × 2½ = d_r = depth between rivet lines
58,500# = 6500 × 9 = V for 1st panel = maximum end shear
10,500# = 6500 × 19.5 ÷ 12 decrease in shear per panel of 19.5″
48,000# = 58,500 − 10,500 = V for 2nd panel
37,500# = 48,000 − 10,500 = V " 3rd "
27,000# = 37,500 − 10,500 = V " 4th "
2¼″ = 6750 × 19.5 ÷ 58,500 = pitch in 1st panel
2¾″ = 6750 × 19.5 ÷ 48,000 = " " 2nd "
3½″ = 6750 × 19.5 ÷ 37,500 = " " 3rd "
5″ = 6750 × 19.5 ÷ 27,000 = " " 4th "
6″ = maximum pitch for balance.

In this first problem each step is clearly indicated, but in general the solution can be simplified by being arranged in tabular form as in subsequent problems. Since the pitch in the first panel is not less than the minimum value 2¼″ found in the first table on page 306, the solution is satisfactory.

CASE III — COMBINED CONCENTRATED AND UNIFORMLY DISTRIBUTED STATIC LOADS APPLIED TO THE WEB

2. **Method.** — The rivet pitches in girders which support both concentrated and uniformly distributed static loads are determined from a combination of the methods of Cases I and II. The pitch for any panel is found from the formula $p = \frac{rd_r}{V}$, in which V is the *total* shear for a section at the beginning of the panel due to the combined loads. If t shear due to the uniform load is relatively small, the pitch is usua changed only at the points of concentration. For example, the fl beam of a double-track through railroad bridge would support four co centrated loads at the stringer points. These concentrated loads wou include the live load, the impact, and the dead load due to the track a the stringers. The uniformly distributed load would consist merely the weight of the floor beam itself which is so comparatively small th the shear is virtually constant between points of concentration. T first pitch would be determined from the total end shear and used fr the end to the first stringer. The second pitch would be determin from the total shear for a section just inside the first concentrated lo and used from the first to the second stringers. The shear in the cent panel would be so small in this case that the maximum pitch of 6″ wou be sufficient. Again, if the concentrated loads are relatively small, t change in shear at the points of concentration may be negligible, and t pitch may be changed only at the panel points selected for the unifo loads, although the shears for sections at these points must include bo systems of loads. Since the rivet pitch increases toward the center, is safe to extend any pitch nearer the center than is theoretically nec sary. When neither system of loads predominates, the pitch should changed both at the points of concentration and at the panels of the u form loads. Each pitch is calculated at one point from the combin shear and extends to the next point where the pitch is found. For e ample, if a concentrated load falls between panel points 2 and 3, t pitch found at 2 would be used between 2 and the load, but from the lo to 3 the pitch found from the shear for a section just to the right of t load would be used. The pitch found at 3 would be used from 3 to unless there was another concentrated load between them, in which ca the pitch would change at the load as before.

3. **Variable Fixed Loads.** — For loads which are fixed in position b variable in magnitude, the pitch in any panel should be determined fr the loads which cause a maximum shear in that panel. This is illustrat by the problem on page 250 : 3.

4. **Illustrative Problem** — *Combined Loads.* — Find the pitches the ¾″ flange rivets in an 80-ft. girder composed of an 84 × $\frac{7}{16}$″ w

6 × 6 angles, and cover plates. There is a concentrated load of 30,000# every 10 feet, and a total uniformly distributed load of 2000#/ft., including the weight of the girder. Use a unit stress in bearing of 20,000#/sq. in.

$$6560\# = r = \text{value of one } \tfrac{3}{4}'' \text{ rivet bearing in a } \tfrac{7}{16}'' \text{ plate (page 309)}$$

$$77.25'' = 84\tfrac{1}{2} - (2 \times 2\tfrac{1}{2} + 2\tfrac{1}{4}) = d_r$$

$$6'\,6'' = B = \text{panel length, approximately equal to } D_r$$

The pitch changes at these panel points and at the points of application of the concentrated loads, as indicated in Fig. 247. The first pitch is used from the end to point 1, the next from 1 to 2, and the last from 5 to the center, being the maximum allowed (page 69 : 1).

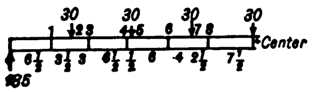


Fig. 247.

Point	Shear = V (in thousands)	Pitch = $\frac{6.56 \times 77.25}{V}$
End	$185.0 = 30 \times 3\frac{1}{2} + 2 \times 40$	$2\frac{3}{4}''$
1	$172.0 = 185.0 - 2 \times 6\frac{1}{2}$	$3''$
2	$135.0 = 172.0 - 30 - 2 \times 3\frac{1}{2}$	$3\frac{3}{4}''$
3	$129.0 = 135.0 - 2 \times 3$	$4''$
4	$116.0 = 129.0 - 2 \times 6\frac{1}{2}$	$4\frac{1}{4}''$
5	$85.0 = 116.0 - 30 - 2 \times \frac{1}{2}$	$6''$ max.

The first pitch is not less than the minimum of $1\frac{1}{16}''$ found in the bottom table on page 306 or the minimum of $1\frac{7}{8}''$ found from the diagram on page 305 so the solution is satisfactory.

CASE IV — MOVING LOADS APPLIED TO THE WEB

1. **How Applied.** — Moving loads may be applied to the webs of the girders of a through bridge by means of some system of solid floor construction in which the floor is connected directly to the girder webs without the use of floor beams.

2. **Theory.** — From the formula $p = \frac{rd_r}{V}$ it is apparent that the rivet pitch varies inversely with the shear, and is therefore minimum when the shear is maximum. That this is true may be shown in two ways. From the derivation of the formula by the shear theory (page 244 : 1) it is obvious that the maximum increase in flange stress will be maximum when the intensity of horizontal shear is maximum, and this will be when the vertical shear is maximum. From the derivation by the bending moment theory (page 243 : 3) it is shown that the increase in flange stress is a function of the vertical shear and is therefore a maximum at the same time. In order to determine the rivet pitch at any point in the flange of a girder, the moving loads should be placed to cause a maximum shear for a section taken at that point. The position of moving loads which will cause a maximum shear for any section is explained on page 189 : 3. The girder is subdivided into panels as in Case II, page 244 : 2. The effect of spacing the rivets according to the maximum shear for moving uniformly distributed loads is shown graphically in Fig. 245. The actual curve of maximum shear is a parabola (8) with the vertex at the support where the shear is zero. The dot and dash stepped line (9) is based upon a constant shear in each panel equal to the maximum shear at the beginning of the panel. The dot and dash line (4) shows the corresponding flange stress. Although there is a considerable excess in the total number of rivets due to the fact that constant pitches are used for practical reasons, yet it is essential that the rivets be placed closely enough at every point to provide for the maximum local stress, even though not all the remaining rivets are fully stressed at the same time. The moving loads must be placed in a different position in determining the shear for the pitch in each different panel, but the shear must be the total shear due to both the moving or "live" loads with impact, and the static or "dead" loads. The latter usually extend the entire length of the girder at all times.

3. **Approximate Method.** — For uniformly distributed live loads, the maximum shear varies with the square of the distance from the section to the vertex of the parabola at the further end of the span, and it is convenient to find the shear for the section at the beginning of each panel from the maximum live-load end shear by proportion in this way. These

shears may be combined with the corresponding dead-load shears to give the proper totals. In case the uniformly distributed live loads are relatively very large, as for example, when there is no other dead load than the weight of the girder, it is usually sufficiently close to apply this ratio to the combined live- and dead-load shears. In this case the inverse ratios may be applied directly to the rivet pitches, and after the pitch in the end panel is determined, the remaining pitches may be found by means of a slide rule without the corresponding shears being found. This approximate solution for uniformly distributed loads gives safe results even though there is additional dead load, but enough rivets may be saved to justify computing the live and dead load shears separately. Sometimes this approximate method, or some similar method, is applied to girders which support moving concentrated loads, but this is not recommended. The shears thus found are less than the shears computed separately, and consequently the resulting pitches are larger than they should be. This difference in pitch in some panels of deck girders under Cooper's engine loads often reaches $\frac{3}{4}''$. In view of this it seems best to compute the maximum total shears for each panel by means of the diagram, in much the same manner as in the problem on page 251 : 1.

1. **Illustrative Problem** — *Moving Uniform Loads.* — Find the rivet pitches in the girder designed on page 222 : 2, using a unit stress in bearing of 24,000#/sq. in.

$9190\# = r =$ value of one $\frac{7}{8}''$ rivet bearing in a $\frac{7}{16}''$ plate (page 310)

$65.25'' = 72\frac{1}{2} - (2 \times 2\frac{1}{2} + 2\frac{1}{4}) = d_r$

$6000\#/\text{ft.} =$ live load

$560\#/\text{ft.} = 200 + 360 =$ dead load

$3040\# = 560 \times 65.25 \div 12 =$ change in dead-load shear per panel length equal to d_r

$11 = 60 \times 12 \div 65.25$ approximate number of panels.

The live-load shear at the beginning of the 2nd panel will then be $\left(\frac{10}{11}\right)^2$ of the live load end shear. This is simpler than using actual lengths in the proportion.

Panel	Shear V (in thousands)			Pitch $= \frac{9.19 \times 65.25}{V}$
	Dead	Live	Total	
End	$17 = .56 \times 30$	$180 = 6.0 \times 30$	197	3
2nd	$14 = 17 - 3$	$149 = 180 \times (\frac{10}{11})^2$	163	$3\frac{3}{4}$
3rd	$11 = 14 - 3$	$120 = 180 \times (\frac{9}{11})^2$	131	$4\frac{1}{2}$
4th	$8 = 11 - 3$	$95 = 180 \times (\frac{8}{11})^2$	103	$5\frac{3}{4}$
5th				6 max.

The use of the approximate method illustrated below would give th same results in this case because the total dead load is so small compare to the live load.

$$3.04 = \frac{9190 \times 65.25}{6560 \times 30} = \text{pitch in end panel}$$

$$3\tfrac{3}{4} = 3.04 \times (\tfrac{11}{10})^2 = \text{“ “ 2nd “}$$

$$4\tfrac{1}{2} = 3.04 \times (\tfrac{11}{9})^2 = \text{“ “ 3rd “}$$

$$5\tfrac{3}{4} = 3.04 \times (\tfrac{11}{8})^2 = \text{“ “ 4th “}$$

CASE V — LOADS APPLIED TO THE FLANGE

2. **How Applied.** — One or more stiffening angles should be place under each heavy concentrated load which rests on the top flange c a girder, in order to transmit the load to the web plate. The rivet pitc is then found according to the method of Case III. Obviously, stiffenin angles cannot be used under moving concentrated loads or under eithe static or moving uniformly distributed loads. The only way in whic these flange loads may be transmitted to the web plate is by means c the flange rivets. These rivets, therefore, must resist not only the sam horizontal flange stresses which they would resist in case the same load were applied directly to the web, but they must resist also a vertic stress. This form of loading is very common. For example, the rai of crane runways rest directly upon the tops of the crane runway girder and the tracks of railway bridges rest directly upon the tops of tl stringers of through bridges and the tops of the girders of deck bridge

Similarly, masonry walls are often built upon the tops of girders. The ties of railroad tracks are so close together that they may be considered as uniformly distributed loads. In order that this vertical stress may be properly distributed among the rivets, the pitch should not exceed 4 or 4½ inches; 4 inches is used for crane runway girders, and 4½ for other deck loads.

1. **Theory.** — In order to combine the effects of both horizontal and vertical stresses on the rivets, they must be reduced to a common basis. For convenience, both the horizontal and the vertical components are found in pounds per linear inch of girder, measured horizontally. The resultant stress shows the maximum stress per linear inch to be resisted by the flange rivets. The rivet pitch in any panel is found by dividing the value of one rivet by the proper resultant stress, thus:

$$p = \frac{r}{\text{resultant}}.$$

The horizontal component per linear inch was found by the shear theory on page 244 : 1 to be $\frac{V}{d_r}$. It could be found by the bending moment theory on page 243 : 3 by dividing the stress per panel $\frac{VB}{D_r}$ by the length of the panel in inches, thus: $\frac{VB}{D_r} \div 12B = \frac{V}{12D_r} = \frac{V}{d_r}$. The V and the d_r are found in exactly the same way as if the loads were applied to the web, the shear being the only variable. The vertical component is usually constant throughout the full length of the girder. It may be composed of several parts, including the proper proportion of all loads which stress the rivets vertically, such as the maximum wheel load with impact or the maximum uniform live load, the dead load due to the track or other superimposed loads,* and the weight of the angles and cover plates of the *top* flange of the girder itself. Most of these loads are expressed in pounds per linear foot, from which the vertical component per linear inch may be found by dividing by 12. It would be impossible to transmit the whole of a heavy wheel load to the web through the single rivet directly under the point of contact of the wheel. This is unnecessary because the rail and the top-chord angles acting as beams distribute the load among several rivets. It is customary to consider the load of a crane wheel to be distributed over 30 inches. In railway bridges it is commonly specified that the maximum wheel load is distributed over three ties. This amounts to about 36 inches, as for example, when 8-inch ties are separated by 6-inch spaces. The maximum wheel load of Cooper's loading is one of the heavy drivers except for short spans in which the maximum shears are obtained from the two special loads, in which case one of the special loads is used. The impact should be included; when this depends upon the loaded length of the track it is close enough to use the span length. The resultant stress per linear inch may best be determined graphically, either by means of the diagram on page 312 or by the use of a simple graph constructed for each problem as follows: lay off two lines at right angles to each other, or use two edges of a rectangular sheet of paper; along one line lay off the constant vertical component; along the other line lay off the different horizontal components; the resultants may be scaled without drawing the corresponding diagonal lines.

2. Loads are sometimes **applied to the bottom flange.** If the loads are divided between the two flange angles the problem is the same as for loads applied to the top flange. If, however, the loads are supported by one angle only, the vertical component tends to shear the rivets in single shear, while the horizontal component tends to shear them in double shear, although the bearing value in the web usually determines the limiting value. Obviously the resultant stress cannot be found from these two components as before. Perhaps the simplest treatment is to increase the vertical component by the ratio which the bearing value of a rivet bears to the single shear value, and then proceed as before, using the bearing value for r.

3. **Illustrative Problem** — *Uniform Flange Loads.* — The problem on page 248 : 1 would be modified as follows, if the load were applied to the top flange instead of the web.

* Students should be cautioned against two common mistakes. The weight of track, including ties, service rails, steel and wooden guard rails and fastenings, is usually given in pounds per linear foot of *track*, and should be divided by 2 to give the weight per foot of girder. The weight of rails is given in pounds per *yard* and not pounds per foot.

6000#/ft. = live load
200#/ft. = superimposed dead load
130#/ft. = (360 − 107) ÷ 2 = weight of top angles and cover plates
6330#/ft. = total load per foot supported by the rivets
530#/in. = 6330 ÷ 12 vertical component per linear inch.

The values of $\frac{V}{d_r}$ are found from the total shears given on page 248 : 1.

Panel	$\frac{V}{65.25}$	Resultant $= \sqrt{\left(\frac{V}{65.25}\right)^2 + 530^2}$	Pitch $= \frac{9190}{\text{resultant}}$
End	3020	3070	3
2nd	2500	2560	3½
3rd	2010	2080	4½
4th			4½ max.

1. **Illustrative Problem** — *Concentrated Flange Loads.* — See page 251 : 1.

CASE VI — GIRDERS IN WHICH THE WEB IS CONSIDERED TO RESIST PART OF THE STRESS DUE TO BENDING MOMENT

2. **Method.** — Probably the large majority of girders are designed according to the method of Case B (page 223 : 3) in which the resisting moment of the web plate is considered. In all these girders the rivet pitches should be determined accordingly. In this method of design a portion of the web plate (usually ⅛ of the gross area) is combined with the net area of the flange angles and cover plates to form the flange area which resists the flange stress due to bending moment. That part of the flange stress which is resisted by the web plate requires no rivets. This is analogous to a simple rectangular beam. The flange rivets are required to transmit the remaining stress to the angles and the cover plates, and to provide for any vertical stress which may result from loads applied to the flange. Thus the methods of the preceding cases may all be modified to apply to girders which are designed by the method of Case B by substituting for V a new value V'. This affects the horizontal component of the stress in the rivets, but it does not affect the vertical component which remains the same as before. The new value V' bears the relation to V that the net area of the flange and cover plates bea to the total flange area including one-eighth or other portion of the we plate. Thus in finding the pitch in any panel,

$$V' = V\left(\frac{a'}{a}\right),$$

in which V is the maximum shear used in finding the pitch according t the method of one of the preceding cases, V' is the corresponding valu used in the method of this Case VI, a' is the greatest net area of th flange angles and cover plates of one flange in the given panel, and a i the sum of a' and the part of the web plate considered as flange are usually ⅛ of the gross area. The ratio $\frac{a'}{a}$ is not constant in a girder unle the same flange area is maintained throughout its length. When cove plates are used, the maximum section is furnished near the point of max mum flange stress, but it is customary to cut off each cover plate at point where the reduced area is sufficient to carry the maximum flang stress which can occur at that point, as explained in the next chapte (page 259). Hence the lengths of the cover plates should be determine before the rivet pitches are computed. Since the ratio $\frac{a'}{a}$ increases wit the area, the greatest cross section in any panel should be used; thi will be found at the end of the panel nearer the center of the girder. I it is not feasible to determine the lengths of the cover plates before th rivet pitches are found, the maximum ratio found from the greates flange area should be used throughout the whole girder.

It is usually better to maintain the same pitch throughout a panel rather tha to change it at the end of a cover plate because such a change would result in th use of a few spaces at a pitch smaller than the pitch either to the right or to the lef This should be avoided for the sake of appearance, particularly as long as very fe rivets, if any, would be saved. If this greatest section extends for only a very sma proportion of the panel length, the smaller area may be used, except for fixed co centrated loads, because the shear at that end of the panel is less than at the b ginning of the panel.

3. **Illustrative Problem** — *Web Loads.* — Find the rivet pitches i the girder designed on page 225 : 1. The concentrated loads found o

page 225 : 1 are placed according to Fig. 251 (a) to give the maximum bending moment. This position also gives the maximum shear in the second panel = 84.2 thousand pounds = 164.3 − 80.1. The maximum shear in the end panel is 194.7 thousand pounds, obtained when

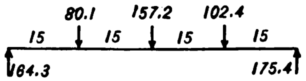


Fig. 251 (a).

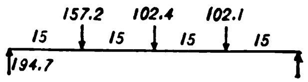


Fig. 251 (b).

the loads are placed to give the maximum bending moment at the end of the first panel, which brings the maximum concentration at the quarter point, as shown in Fig. 251 (b).

$9190\# = r =$ value of one $\frac{7}{8}''$ rivet bearing in a $\frac{7}{16}''$ plate at 24,000#/sq. in.

$65.25'' = 72\frac{1}{2} - (2 \times 2\frac{1}{2} + 2\frac{1}{4}) = d_r$

$12{,}300\# = 410 \times 30 =$ end shear due to weight of girder

$6{,}200\# = 12{,}300 - 410 \times 15$ corresponding shear for second panel

$$\frac{37.1}{41.1} = \frac{13.9 + 8.2 + 7.5 + 7.5}{3.9 + 13.9 + 8.2 + 7.5 + 7.5} = \text{ratio } \frac{a'}{a} \text{ for 2nd panel}$$

$$\frac{29.6}{33.5} = \frac{37.1 - 7.5}{41.1 - 7.5} = \text{ratio for end panel, see Fig. 262}$$

$$182.9 = \frac{29.6}{33.5}(194.7 + 12.3) = V' \text{ for end panel in thousand pounds}$$

$$81.6 = \frac{37.1}{41.1}(84.2 + 6.2) \quad = V' \text{ `` 2nd `` `` `` ``}$$

$$3\frac{1}{4}'' = \frac{9.19 \times 65.25}{182.9} = \text{pitch in end panel}$$

$$6'' \max = \frac{9.19 \times 65.25}{81.6} = \text{`` `` 2nd `}$$

1. **Illustrative Problem** — *Flange Loads.* — Find the rivet pitches in a stringer for the same bridge as the preceding problem. The stringer is composed of a $22 \times \frac{5}{8}$ web and $6 \times 6 \times \frac{11}{16}$ angles without cover plates.

$13{,}130\# =$ value of one $\frac{7}{8}''$ rivet bearing in a $\frac{5}{8}''$ plate at 24,000#/sq. in.

$15'' = 22\frac{1}{4} - (2 \times 2\frac{1}{2} + 2\frac{1}{4}) = d_r$ (the web being flush with top angles) The panel length is taken equal to d_r. The dead-load end shear is 2800# $= (\frac{450}{2} + 150)\ 7.5$, which is reduced by $500\# = 375 \times 1.25$ per panel. The live-load shears for the first two panels are found by means of the table on page 318. The remaining live-load shears are maximum for the two special 37,500# loads spaced 7 feet apart, since the loaded segment does not exceed 12.5 feet (page 194 : 1). A single impact percentage determined from the span length may be used for each panel. The constant ratio $\frac{a'}{a} = \frac{14.2}{15.9} = \frac{14.2}{1.7 + 14.2}$. The vertical component per linear inch for the two end panels is $1650\# = \frac{30{,}000 \times \frac{615}{315}}{36} + \frac{\frac{450}{2} + 50}{12}$, the 50 being the weight of the top flange angles. The vertical component for the remaining panels is $2050\# = \frac{37{,}500 \times \frac{615}{315}}{36} + \frac{\frac{450}{2} + 50}{12}$.

Panel	Horizontal Component $= \frac{V'}{15}$	Vertical Component	Resultant	Pitch $= \frac{13{,}130}{\text{resultant}}$
End	$7140 = \left(60{,}000 \times \frac{615}{315} + 2800\right)\frac{14.2}{15.9} \div 15$	1650	7330	$1\frac{3}{4}$
2nd	$6240 = \left(52{,}500 \times \frac{615}{315} + 2300\right)\frac{14.2}{15.9} \div 15$	1650	6450	2
3rd	$5330 = \left(45{,}000 \times \frac{615}{315} + 1800\right)\frac{14.2}{15.9} \div 15$	2050	5710	$2\frac{1}{4}$
4th	$4580 = \left(38{,}800 \times \frac{615}{315} + 1300\right)\frac{14.2}{15.9} \div 15$	2050	5020	$2\frac{1}{2}$
5th	$3820 = \left(32{,}500 \times \frac{615}{315} + 800\right)\frac{14.2}{15.9} \div 15$	2050	4340	3
6th	$3080 = \left(26{,}300 \times \frac{615}{315} + 300\right)\frac{14.2}{15.9} \div 15$	2050	3700	$3\frac{1}{2}$

The rivet pitches are here determined up to the center in order to illustrate the method, although as a matter of fact they could not be used because the pitch in the end panel is less than the minimum of $2\frac{1}{4}$ found from the table on page 306. In this case the girder should be redesigned with either an increased web thickness or increased depth.

CASE VII—BOX GIRDERS, CANTILEVER GIRDERS, AND GIRDERS WITH NON-PARALLEL FLANGES

1. **A box girder** may be considered as two or more girders, depending upon the number of webs (Fig. 95 (*g*)). The cross section is usually made symmetrical about a vertical axis. When there are only two webs each half of the girder will carry one-half the load, and one-half the total flange stress will be resisted by the angle (or angles) on each web together with one-half the cover plate. The flange rivets in each web must be proportioned for this half stress, according to the method of one of the preceding cases. If flange angles are used on only one side of each web, the rivets will be in single shear, and the corresponding rivet value must be chosen. Girders with three webs are usually designed so that the center portion carries one-half the total load. Thus there are two angles in each flange on the central web, but only one on each outer web. The central web should be thick enough to make the value of one rivet twice the single-shear value so that the rivet pitches in all three webs will be the same. Otherwise, the pitches must be determined separately.

2. In **a cantilever girder** the rivet pitches may be determined by the method of one of the preceding cases given for simple girders, but care must be taken that the proper shear is used. In determining the pitch in any panel, the maximum shear in the panel should be used. Sometimes this maximum will occur when the section is taken at the left end of the panel and sometimes when it is taken at the right end, depending on the position of the panel in the girder. See page 190 : 1 for the position of moving loads which will cause the maximum shears.

3. **Illustrative Problem** — *Cantilever Girder.* — Find the pitches of ¾″ rivets in a 40-ft. cantilever girder which projects 10 ft. beyond one of two supports which are 30 ft. apart. The total static load is 4,000#/ft. including the weight of the girder. The girder is composed of a 36 × ⅜ web and 6 × 4 angles. The unit stress in bearing is 20,000#/sq. in. Assume the simplest case in which the load is applied to the web, and the resisting moment of the web is neglected.

The shear and moment diagrams for this form of loading are shown in Fig. 193. The maximum bending moment occurs at the section for which the shear is zero, and the point of contraflexure is at the point where the bending moment is zero. The bending moment and shea and hence the rivet spacing, are symmetrical between the point of con traflexure and the supported end. The distance from the right suppo to the point of contraflexure is 26.6 found by equating to zero an expre sion for bending moment in terms of the distance X. The point of contra flexure is not much over a panel length from the left-hand support s the equal panels are laid off from the point of contraflexure and from th right-hand support as shown in Fig. 252 in order to make the rivet spacin symmetrical. If preferred, the equal panels could be laid off from the left-hand support, but additional pitches would have to be calculated because the shears for the panels at the right would differ fron those at the left. The cantilever end is divided into four equal panels.

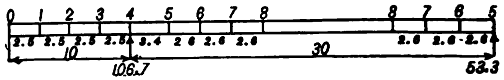


Fig. 252.

$$5630\# = r = \text{value of one } \tfrac{3}{4}'' \text{ rivet bearing in a } \tfrac{3}{8}'' \text{ plate}$$
$$31.5'' = 36\tfrac{1}{2} - 2 \times 2\tfrac{1}{2} = d_r.$$

Panel	Shear = V (in thousands)	Pitch = $\frac{5.63 \times 31.5}{V}$
0–1	10.0 = 4.0 × 2.5	6 max.
1–2	20.0 = 10.0 × 2	6 max.
2–3	30.0 = 10.0 × 3	6
3–4	40.0 = 10.0 × 4	4½
4–5	66.7 = 106.7 − 40.0	2¾
5–6	53.3 = R_R	3¼
6–7	42.9 = 53.3 − 4.0 × 2.6	4¼
7–8	32.5 = 42.9 − 4.0 × 2.6	5½
8–8	22.1 = 32.5 − 4.0 × 2.6	6 max.

The smallest space 2¾ is not less than the minimum $2\tfrac{7}{16}$ found in th bottom table on page 306 so the solution is satisfactory.

4. **Girders with non-parallel flanges** are used for turntables, sidewall brackets, and other special work. The inclined flanges have vertica components which relieve the web plate of part of the shearing stresses and the number of rivets must be increased accordingly. The formul

$p = \frac{rd_r}{V}$, or a modification of it for conditions similar to those of the preceding cases, may be adapted to girders with inclined flanges by substituting V'' for V, as explained below. The resulting pitch will be the distance *along the flange* and not necessarily horizontal. The pitch along the bottom flange will theoretically be the same as the pitch along the top flange even though the inclination is not the same. Sometimes for practical reasons, only the pitch of the rivets in the steeper flange is computed, the rivets in the other flange being placed in the same vertical lines even though more rivets are thus used. The pitch is usually changed at equal intervals. Since both V'' and d_r vary, the pitches should be determined at both ends of a panel and the smaller of the two should be used throughout that panel. The value of V'' may be determined from the following expression: *

$$V'' = V - \frac{M}{d_r}(\tan \alpha + \tan \beta)$$

in which V = the maximum shear for a given section, M = the corresponding bending moment for the same section and for the same position of the loads, d_r = the vertical distance between rivet lines at the given section, α and β = the angles of inclination of the bottom and the top flanges with the horizontal. The signs of the tangents given above are based upon the flanges converging toward the end of the girder. In case either slope is reversed the corresponding sign of the tangent should be changed. If either flange is horizontal the corresponding tangent becomes zero.

CASE VIII—GIRDERS WITH VERTICAL FLANGE PLATES, AND GIRDERS WITH FOUR ANGLES IN EACH FLANGE

1. **When Used.** — Girders which carry heavy loads over long spans frequently require greater flange areas than can be furnished by two angles with cover plates. Similarly the use of a depth less than the most economical depth may necessitate the use of a more complex flange. Various forms of flange are adopted to meet different requirements, as illustrated in books on girder design. Only two forms will be considered here, viz.: flanges with vertical plates between the angles and the web, and flanges composed of four angles. Vertical flange plates are often used in conjunction with the four angles, either between the angles and the web or outside of the vertical legs of the angles, or both, but any draftsman who is likely to design the details for such a girder should be able to adapt the principles of this chapter to his needs.

2. **Vertical flange plates** which are placed between the flange angles and the web plate extend the entire length of the girder because it is impractical to make them shorter than the angles which rest upon them. The rivets which pass through both the angles and the vertical plates (rows a and b, Fig. 253) must be considered separately from those which pass through the web and the vertical plates only (row c). The former transmit only that portion of the flange stress which is carried by the flange angles and the cover plates. This part of the stress tends to shear the rivets between the angles and the vertical plates, and the thickness of metal is sufficient to develop the full double-shear value of the rivets. The total stress carried by the angles, the cover plates, and the vertical flange plates tends to shear the rivets in all rows between the vertical plates and the web, and the rivet value is usually limited by the bearing value in the web plate. For practical reasons it is customary to place the rivets in row c opposite those in rows a and b. The pitch in row c increases more rapidly than the pitch in rows a and b. If the pitch in rows a and b is found to be less than that in row c in the end panel it will also be less in the other panels, so it is unnecessary to determine the pitch in row c in every panel. Sometimes the pitch in row c will be twice as great as in rows a and b in some panels, and the rivets may be placed opposite those in row a only, as in Fig. 253, provided the

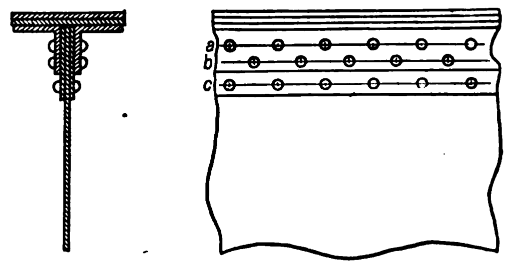


Fig. 253.

* For derivation see Johnson-Bryan-Turneaure's "Modern Framed Structures," Vol. III, John Wiley and Sons, Inc., New York; or Waddell's "Bridge Engineering," Vol. I, John Wiley and Sons, Inc., New York.

maximum value (usually 6″) is not exceeded. The pitch in row c must then be determined in enough panels to find where the double pitch may begin.

1. **Theory** — *Rivets in Angles.* — The rivet pitch in rows a and b is determined from the expression $p = \frac{r''d_r}{V'''}$, in which r'' = the double shear value of one rivet, d_r = the mean depth between the rivet lines in the angles (rows a and b), and V''' = that proportion of the maximum total shear V in the panel which the net area of the angles and the cover plates bears to the total net flange area in the panel, including the vertical flange plates. Since this proportion increases with the areas, both areas should be taken for a section at the end of the panel toward the center of the girder, although the maximum shear is not found for the same section (compare page 244 : 3). The resulting pitch is the horizontal distance from a rivet on row a to a rivet on row b, the rivets being staggered. If three rows are used in each angle, the rivets in the middle row are usually staggered with the rivets in the top and bottom rows which are placed opposite. The pitch from a rivet in the middle row to the rivets in the other rows would be three-halves of the pitch found from the formula, d_r being the mean depth of the three rows. If the loads are applied to the flange of the girder, a vertical component must be combined with the horizontal component as in the method of Case V (page 248 : 2). If the girder is designed according to Case B (page 223 : 3), the total net flange area mentioned above should include one-eighth or other portion of the web which is counted as flange area, as explained under Case VI (page 250 : 2).

2. **Theory** — *Rivets in Vertical Plates.* — The determination of the pitch of the rivets in row c (Fig. 253) is based upon the difference between the total number of rivets required in rows a, b, and c, and the number required in rows a and b. The total number of rivets in all three rows required in a panel of B feet is $\frac{VB}{rd'}$ (compare page 243 : 3), in which V = the maximum total shear in the panel, r = the value of one rivet in bearing in the web (or double shear if less), and d' = the mean depth of the three rows of rivets or $\frac{aa + bb + cc}{3}$. The corresponding number of rivets in rows a and b is $\frac{V'''B}{r''d_r}$ (preceding paragraph). The number of rivets in row c is equal to the difference between these two numbers, and the rivet pitch is found by dividing the panel length by this difference. The panel length cancels out and the formula for the rivet pitch in row c is

$$p = \frac{1}{\frac{V}{rd'} - \frac{V'''}{r''d_r}}.$$

If the loads are applied to the flange of the girder, the same vertical component used in finding the pitch in rows a and b should be combined with $\frac{V}{d'}$ and also with $\frac{V'''}{d_r}$ to give two resultants which should be used in place of the corresponding quantities in the above formula. If the girder is designed according to Case B, the V of the formula should be replaced by V' as in Case VI (page 250 : 2) and the V''' should be modified as in the preceding paragraph. In case a deeper vertical plate is used with an additional row of rivets, the rivets would be staggered on the two lines, and the pitch of the staggered rivets would be found as before except the extra rivet line would have to be considered in determining the mean depth d'.

3. **Illustrative Problem** — *Girder with Vertical Flange Plates.* — The points peculiar to this type of girder may be illustrated by finding the pitch in one panel where the maximum shear = 600,000#. The girder is composed of a 120 × $\frac{5}{8}$ web, 6 × 6 angles, 14″ cover plates, and 12 × $\frac{5}{8}$″ vertical flange plates, with $\frac{7}{8}$″ rivets. Let us assume that the resisting moment of the web is considered; that the ratio of the net area of the angles, cover plates, and vertical plates to the total net area including $\frac{1}{8}$ the web is 0.8, or $V' = 0.8V$; and that the ratio of the net area of the angles and cover plates to this same total net area is 0.6, or $V''' = 0.6V$. The depth back to back of angles is 10′ 0$\frac{1}{2}$″, the distance from the back of the angle to the first row of rivets a is 2$\frac{1}{2}$, from row a to row b is 2$\frac{1}{4}$, from row b to row c is 3, and from row c to row d (the additional row in the vertical plates) is 3. The mean depth d_r of the rows a and b is 113$\frac{1}{4}$, and the mean depth d' of rows a, b, c, and d is 107.6. From the last table on page 310, the value of one rivet in double shear r'' = 12,030, and

in bearing in the $\frac{5}{8}''$ web plate $r = 10{,}940$. The pitch of the staggered rivets in rows a and b is $3\frac{3}{4} = \frac{12{,}030 \times 113.25}{0.6 \times 600{,}000}$. The pitch of the staggered rivets in rows c and d is calculated to be over the maximum of six inches, thus:

$$\frac{1}{\dfrac{0.8 \times 600{,}000}{10{,}940 \times 107.6} - \dfrac{0.6 \times 600{,}000}{12{,}030 \times 113.25}}.$$

Since $6''$ is less than $2 \times 3\frac{3}{4}$ the double space cannot be used, and for practical reasons a pitch of $3\frac{3}{4}$ would be used, with the rivets in rows c and d opposite those in rows a and b. In case the load was applied to the flange and the vertical component was 1000# per linear inch of girder, the above problem would be modified as follows. The horizontal component $\frac{V'''}{d_r} = 3280 = \frac{0.6 \times 600{,}000}{113.25}$ must be replaced by the resultant of this force and the vertical component of 1000, or 3430. The horizontal component $\frac{V'}{d'} = 4460 = \frac{0.8 \times 600{,}000}{107.6}$ must be replaced by the resultant of this force and 1000, or 4750. The rivet pitch in rows a and b would then be $3\frac{1}{2} = \frac{12{,}030}{3430}$, and the pitch in rows c and d would become $\frac{1}{\frac{4570}{10{,}940} - \frac{3430}{12{,}030}}$ which is over $6''$. A pitch of $3\frac{1}{2}$ would be used since $6''$ is less than $2 \times 3\frac{1}{2}$.

1. When **four angles in each flange** are used, the rivets in the additional angles are placed opposite those in the outer angles, as shown in Fig. 255, for convenience in spacing the rivets both in the drafting room and in the shop. It is therefore necessary only to determine the pitches in either the outer or the inner angles whichever is smaller. The rivets in the outer angles are proportioned for the entire vertical component due to any load which rests upon these angles, and for that portion of the horizontal stress which is carried by the outer angles and the cover plates. The rivets in the inner angles are proportioned simply for that part of the horizontal stress which is carried by the inner angles (unless there should be a vertical load applied to these angles). The depth d_r should be the mean depth between the rivet lines in the angles in which the pitch is determined, the mean depth for the inner angles being considerably less than that for the outer angles. When cover plates are used, the limiting pitch is found in the outer angles, otherwise the pitches in both angles should be determined in one or more panels and the smaller values chosen.

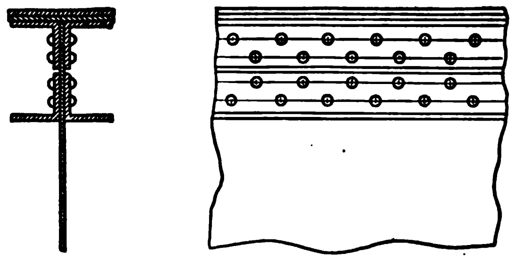


Fig. 255.

THE DETERMINATION OF THE MINIMUM PITCH BASED UPON THE STRENGTH OF THE WEB

2. **Importance.** — One point of failure of plate girders is apparently overlooked by some designers, although it should receive the most careful consideration. The strength of the web plate between flange rivets may not be sufficient to transmit the full flange stress for which the flanges are designed and for which the rivet pitches are determined. If the rivet pitch which is determined by any of the methods of the preceding Cases happens to be less than a certain minimum, the web plate will fail along the line of rivets before the strength of the rivets is fully developed, and the maximum safe load of the girder will be somewhat less than the required amount. The minimum space of "three diameters" (page 68 : 6) is so generally recognized as the smallest pitch allowed for rivets in a single line that no serious difficulty is likely to arise from the lack of further consideration of flange rivets which are placed in a single line, although the minimum pitch is not *exactly* three times the diameter of the rivet for all unit stresses. A draftsman is quite liable, however, to use a smaller pitch for staggered rivets than is justified by the strength of the web plate. Although the minimum pitch for staggered rivets is somewhat less than "three diameters," it is not so much less as many designers and draftsmen suppose. It is important that no

pitch be less than the minimum determined by the strength of the web as explained below.

1. **A table of minimum pitches** for flange rivets is given on page 306. Values are shown for different sizes of rivets, for different web thicknesses, and for different unit stresses. Distinction is made also between rivets placed in single and double rows, and between those which act in single and double shear. It should be noted that the pitch varies with the web thickness only when the rivet value is limited by either single or double shear, but the pitch is independent of the web thickness when the rivets are limited by the bearing value as is more frequently the case. Values are given in the tables only for webs from $\frac{5}{16}''$ to $\frac{3}{4}''$ thick, and no value is given which would provide less than the minimum clearance required in driving the rivets by machine. Since rivets are so commonly staggered on lines which are $2\frac{1}{4}''$ apart it has seemed desirable to indicate by italics all values which are less than the minimum determined for this gage from the diagram on the page preceding the table. These values in italics should be used only upon the assurance that the corresponding reduction in the net flange area will not impair the strength of the girder. For example, this is true when rivets in both lines have been deducted in determining the net area used in the design of the flange, or when there is sufficient excess of flange area in the panel for which the pitch is determined as is quite often the case.

2. **Unit Stresses.** — The unit stress in shear is usually specified for the gross area of the web plate but seldom for the net area. In determining the strength of the web between rivets the net area is used. In the tables two different unit stresses in shear on the net section of the web are used in conjunction with the common unit stresses for the rivets. Unless otherwise specified a unit stress of 13,000#/sq. in. may be used. This is about eight-tenths of the unit stress in tension of 16,000, which is a fair allowance. It is also about four-thirds of the unit stress in shear on the gross section of 10,000, which is consistent with the ratio used in allowing for rivets in the web plate in deriving the formula used for designing girders by the method of Case B (see page 223: 3). Incidentally, the use of 13,000 with the unit stresses for rivets as specified by the American Railway Engineering Association results in a minimum pitch in a single row of rivets of three diameters, as shown in the first table on page 306.

3. **Theory.** — The web plate must be strong enough to transmit the flange stress to the flange angles. The critical horizontal section is along the line of flange rivets. When two lines of rivets are used, the critical section will be along the line nearer the center of the web. The web between adjacent rivets on the critical line must develop the strength of the corresponding number of rivets, i.e., one when a single line is used as in Fig. 257 (*a*) and two when the rivets are staggered as in Fig. 257 (*b*). The formulas upon which the values in the tables on page 306 are based depend upon whether rivet values are limited by bearing, or by single or double shear. The formulas for the minimum pitch for rivets in a single line and for staggered rivets are so similar that they are derived in parallel columns for the sake of comparison. Similar expressions for three or more lines of rivets may be derived in like manner as occasion demands. Let p = the minimum rivet pitch, measured parallel to the rivet lines from center to center of rivets, d = the diameter of the rivet, $d + \frac{1}{8}$ = the diameter of the rivet hole used in designing (page 208 : 2), t = the thickness of the web plate (or the thickness of a single angle of a box girder if less than the web thickness), s = the unit stress in shear on the net section of the web plate, s' = the unit stress in shear on the rivets, and b = the unit stress in bearing. All units are inches or pounds per square inch.

RIVETS IN SINGLE LINE

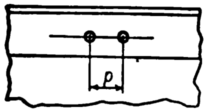


Fig. 257 (*a*).

In Fig. 257 (*a*), the net area which resists horizontal shear in the space p is $[p - (d + \frac{1}{8})]t$. This is multiplied by the unit stress s and equated to the value of *one* rivet.

If the rivets are limited by the bearing value:

$$\left[p - \left(d + \frac{1}{8}\right)\right]ts = dtb, \quad \text{or} \quad p = \frac{db}{s} + \left(d + \frac{1}{8}\right).$$

If the rivets are limited by the single shear value:

$$\left[p - \left(d + \frac{1}{8}\right)\right]ts = \frac{\pi d^2 s'}{4}, \quad \text{or} \quad p = \frac{\pi d^2 s'}{4ts} + \left(d + \frac{1}{8}\right).$$

If the rivets are limited by the double shear value:

$$\left[p - \left(d + \frac{1}{8}\right)\right]ts = \frac{2\pi d^2 s'}{4}, \quad \text{or} \quad p = \frac{\pi d^2 s'}{2ts} + \left(d + \frac{1}{8}\right).$$

STAGGERED RIVETS

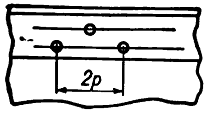


Fig. 257 (*b*).

In Fig. 257 (*b*), the net area which resists horizontal shear in the space $2p$ is $[2p - (d + \frac{1}{8})]t$. This is multiplied by the unit stress s and equated to the value of *two* rivets.

If the rivets are limited by the bearing value:

$$\left[2p - \left(d + \frac{1}{8}\right)\right]ts = 2dtb, \quad \text{or} \quad p = \frac{db}{s} + \frac{1}{2}\left(d + \frac{1}{8}\right).$$

If the rivets are limited by the single shear value:

$$\left[2p - \left(d + \frac{1}{8}\right)\right]ts = \frac{2\pi d^2 s'}{4}, \quad \text{or} \quad p = \frac{\pi d^2 s'}{4ts} + \frac{1}{2}\left(d + \frac{1}{8}\right).$$

If the rivets are limited by the double shear value:

$$\left[2p - \left(d + \frac{1}{8}\right)\right]ts = \frac{2 \times 2\pi d^2 s'}{4}, \quad \text{or} \quad p = \frac{d^2 s'}{2ts} + \frac{1}{2}\left(d + \frac{1}{8}\right).$$

SUMMARY

CASES I, II, III, and IV. Girders in Which the Loads are Applied to the Web and the Resisting Moment of the Web is Neglected. — The maximum pitch in each panel is found from $p = \frac{rd_r}{V}$ in which r = the limiting value of one rivet, d_r = the mean depth between the rivet lines of the top flange and those of the bottom flange, and V = the *maximum* total shear at the beginning of the panel. No pitch should be more than 6″ or less than the minimum found from the table on page 306. The panel lengths are chosen approximately equal to the depth between rivet lines, although this may be varied somewhat to conform to the spacing of the stiffening angles. The pitch is also changed at the point of application of any fixed concentrated load. If there are no other loads besides the fixed concentrated loads except the weight of the girder, the pitch is changed only at these points of concentration. For girders which support moving uniformly distributed loads the intermediate pitches may be found from the pitch in the end panel as explained on page 247 : 3.

CASE V. Girders in Which the Loads are Applied to the Flange and the Resisting Moment of the Web is Neglected. — The maximum pitch in each panel is found from $p = \frac{r}{\text{resultant}}$, in which the resultant is found from horizontal and vertical components in pounds per linear inch. The horizontal component is $\frac{V}{d_r}$ found from the same values and at the same points as in Cases I–IV above. The vertical component is found from the loads which tend to shear the rivets vertically, as explained on page 249 : 1. No pitch should exceed $4\frac{1}{2}$″ (usually 4″ for crane runway girders), or be less than the minimum found from the table on page 306.

CASE VI. Girders in Which the Web is Considered to Resist Part of the Flange Stress due to Bending Moment. — The methods of Cases I to V inclusive may be made to apply to girders designed by the method of Case B (page 223 : 3) by simply multiplying each value of V by the ratio $\frac{a'}{a}$, where a' is the greatest net area of the flange angles and cover plates in a given panel and a is the sum of a' and that portion of the web plate considered as flange area (usually $\frac{1}{8}$ of the gross area).

CASE VII. Box Girders; see page 252 : 1. **Cantilever Girders;** see page 252 : 2. **Girders with Non-parallel Flanges;** see page 252 : 4.

CASE VIII. Girders with Vertical Flange Plates; see page 253 : 2. **Girder with Four Angles in Each Flange;** see page 255 : 1.

CHAPTER XXXVIII

COVER PLATES

SYNOPSIS: The methods of finding the lengths of the cover plates of plate girders to be used under different conditions, and the determination of the rivet spacing in the cover plates.

1. **Use.** — Cover plates are often riveted to the flange angles of plate girders in order to furnish the necessary flange area when the angles alone are insufficient, as explained on page 219: 1. The use of cover plates which do not extend the full length of the girder permits the reduction of flange area toward the ends of the girders to correspond to the reduction in the flange stress due to bending moment.

LENGTHS OF COVER PLATES

2. **Some cover plates extend the full length** of the girder in order to furnish protection from the weather, or to give uniform bearing. On girders which are exposed to the elements, one cover plate of the top flange is made to extend the entire length to prevent the collection of water in the pocket formed between the angles above the edge of the web plate (page 95: 3). Some specifications require that one of the bottom plates extend the full length also, particularly in bridges over salt water. This keeps the girder symmetrical about the neutral plane, although no harm can come from having an excess in one flange even though it is not balanced by an excess in the other. In crane runways the rails usually rest directly upon the top flanges of the girders, and in order to give proper bearing for the rails all the top cover plates, or the cover channel (Fig. 95 (*e*)), must run the full length. This is not required in the top flanges of deck railway bridge girders because the ties can be notched to make up for the variation in the thickness of the plates.

3. **The theoretical length** of a cover plate is determined from the curve of bending moments, but the actual length is usually made from two to three feet greater. (Compare pages 261: 2 and 262: 2.) This extension of one foot or more at each end permits the partial development of the plate by rivets beyond the point where the plate theoretically begins, thus insuring its action when needed. The extension also provides a safe margin for any inaccuracies in length due to scaling the graph or to calculation, and minimizes the chance of the girder failing at the end of a cover plate.

4. **The graphic method** of determining the theoretical lengths of cover plates will be explained first because it is more general in that it can be adapted to any condition of loading. **The algebraic method** is more convenient than the graphic method upon which it is based, but it is limited to uniformly distributed or moving loads, as explained on page 263: 1.

5. For girders which are **symmetrically loaded** only one-half of the span need be plotted. This is true also for girders with unsymmetrical loads which may be reversed, as in bridges. For unsymmetrical fixed loads the full length should be plotted. Usually the curve of bending moments for a single position of the loads is sufficient, but for variable fixed concentrated loads more than one curve may be required, as explained on page 261: 2.

6. The bending moments for a girder which supports a **uniformly distributed load** vary as the squares of the distances, and the curve of moments is a parabola. This is true for moving loads as well as for static

loads because the maximum bending moment at any point occurs when the load extends the entire length of the girder. The vertex of the parabola is at the center of the span where the bending moment is maximum. To any convenient scale, AB equal to the half span (or full span as explained below) is laid off horizontally, and BC equal to the total maximum bending moment is laid off vertically, as in Fig. 260. The vertical scale is different from the horizontal scale because the units are different. It is well to choose the scales so that the distance BC is no greater than

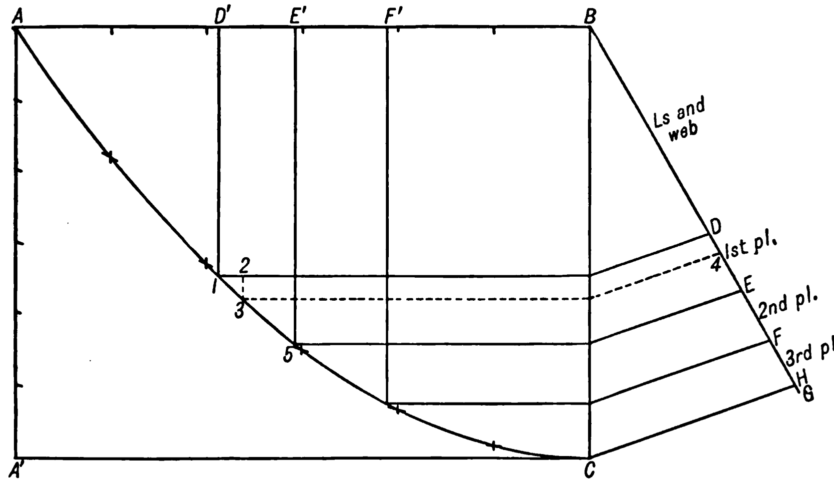


Fig. 260. Lengths of Cover Plates — Uniformly Distributed Loads.

the distance AB and no less than three-quarters of AB. With the vertex at C a parabola* is drawn through A. The ordinate to the curve from any point between A and B represents the bending moment at that point. On any inclined line through B is laid off BH equal to the total *required* net flange area for which the girder is designed. This is subdivided so that BD is the actual net area of the two flange angles together with the portion of the web, if any, which is counted as flange area in the design, and DE, EF, FG, etc., are the net areas of the cover plates, the larger being nearer the angles. Usually the last point will fall a little outside of the point H because of the excess in area which results from the selection of commercial sizes. A line is drawn from H to C, and lines parallel to this line are drawn through points D, E, and F. These lines make proportionate intercepts on the line BC which show the portion of the maximum bending moment which is resisted by each of the component parts, since the net areas are directly proportional to the bending moments (page 221: 2). Horizontal lines drawn through these new points cut the parabola at points where the corresponding net areas satisfy the bending moment. Thus, from A to D' the web and the flange angles are sufficient without cover plates; at D' the first cover plate becomes necessary, but it is not fully developed until E' is reached, at which point the second cover plate begins; the third plate begins at F', etc. The theoretical length of the first (thickest) plate is twice the distance from D' to B, the second twice the distance $E'B$, and the third twice $F'B$. By making AB equal to the whole span instead of one-half, the effect will be to change the scale so that the distances $D'B$, $E'B$, and $F'B$ give the whole lengths of the plates without doubling. If the line BH is drawn so that the line HC will be horizontal, the parallels will be coincident with the horizontal lines which cut the curve, and the construction is simplified (see illustrative problem, page 262: 1). It is often more convenient not to plot the bending moment, but to lay off the net areas along the line BC and to draw the parabola through the new point $C = H$. The curve then represents net areas instead of bending moments, and the inclined line is not needed. It is quite a common practice to draw the parabola to correspond to the point G of the actual area instead of point H of the required area. This gives safe results but inconsistent results, because the lengths of some cover plates may be increased a foot or more while others in the same girder are changed only slightly. It is unnecessary to strengthen the flange throughout its length simply because it happens to be impossible to find a commercial size which will satisfy the requirements at the center. If all other parts of the girder

* Construction: Draw a vertical through A, and lay off $AA' = BC$. Divide AA' into any number of equal parts, depending upon the accuracy required. Connect each of these points with C. Divide AB into the *same number* of equal parts and drop a vertical through each point. The intersection of the first vertical with the first diagonal gives the first point on the parabola, etc. The points may be joined by means of a curved ruler. Only enough of the construction lines need be drawn to show the intersections.

were strengthened in proportion it might be desirable, but even then a more equitable distribution could be devised. However, no great harm can come from the use of actual areas instead of the required areas, and in some cases it is more convenient.

1. The bending moments for a system of **concentrated loads** increases constantly between loads, and the curve of moments is a series of straight lines. Since concentrated loads are found only in conjunction with uniformly distributed loads, the two curves must be combined. This may

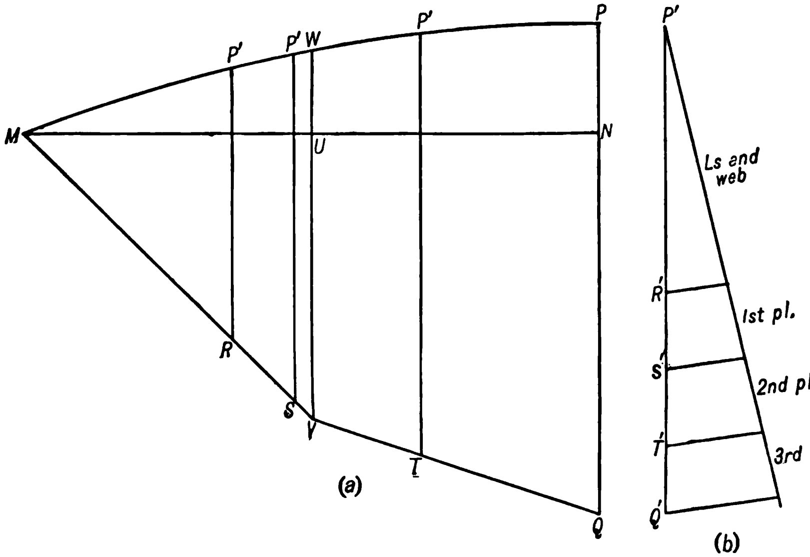


Fig. 261 (*a*). Lengths of Cover Plates — Combined Loads.

be best accomplished by plotting one below and the other above a horizontal line so that the combined ordinates representing the total bending moments may be scaled. Thus in (*a*), Fig. 261 (*a*), the parabola is drawn, through P and M so that NP is the maximum bending moment due to the uniform load, as in the preceding paragraph. The bending moment due to the concentrated loads must be found and plotted at the point of application of each load, as UV and NQ. The total maximum bending moment for which the flanges were designed is represented by the maximum combined ordinate PQ. This total bending moment should be subdivided in proportion to the resisting moments of the angles and web, and each cover plate, as in the preceding paragraph. This should be done along the edge of a separate card or piece of paper, as shown in (*b*), Fig. 261 (*a*). This card can be made to slide over the graph of (*a*), Fig. 261 (*a*), so that P' follows the parabola, and the line $P'Q'$ is kept vertical. In the position where R' falls in the lower curve at R, the vertical line marks the point where the first cover plate should theoretically begin, because the resisting moment of the angles and web $P'R'$ satisfies the total bending moment $P'R$. Similarly, the second cover plate should begin where the total ordinate $P'S$ equals the distance $P'S'$, the third where the ordinate $P'T$ equals $P'T'$, etc. In case there is no other uniform load than the weight of the girder itself, the corresponding bending moment would be relatively so small that the parabola would be too flat to plot. In this event a single curve should be plotted from the *total* bending moments. Thus the bending moment due to the concentrated loads at the point of application of each load should be increased by the bending moment due to the uniform load at the *same* point.

2. For **variable concentrated loads,** fixed in position, the lengths of the cover plates are not always determined from a single position of the loads. Take, for example, the girder of a through railway bridge. The loads should be placed to give the maximum bending moment for which the flanges are designed (page 225 : 1), the total bending moment at each panel point should be found for this position of the loads, and the corresponding curve of bending moments should be plotted, as shown by the full line in Fig. 261 (*b*). The loads should then be placed to cause the maximum bending moment at one of the other panel points, and the curve of

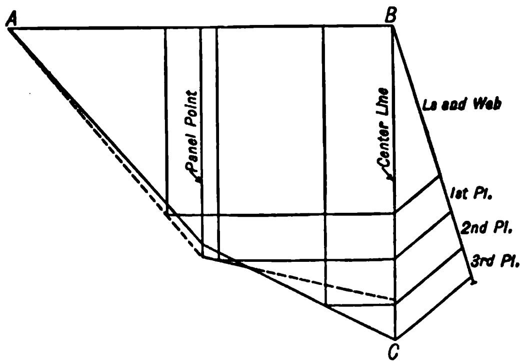


Fig. 261 (*b*). Lengths of Cover Plates — Variable Concentrated Loads.

bending moments should be constructed for this position in the same manner, as represented by the dashed line. Similarly, a curve should be plotted for the position of the loads which causes the maximum bending moment at each other panel point. Since the live loads may cross the bridge in either direction, the girder is made symmetrical and only one-half need be plotted. Each curve should be plotted for that half of the girder in which the larger bending moments occur. Each curve will be outside of all the other curves at least at one panel point. The maximum ordinate BC representing the maximum bending moment for which the flange is designed should be subdivided as before (page 259: 6) to show the proportion of the bending moment which is resisted by each component part. The length of each cover plate is determined by drawing a horizontal line through the proper point on the line BC until it intersects the curve of bending moments which is farthest from the center. The bending moments due to the weight of the girder are usually so small that they can be combined with the bending moments due to the concentrated loads which include other dead loads, live loads, and impact. If there were additional uniformly distributed loads it might become necessary to plot the moment parabola separately, as explained in the preceding paragraph. There are so many steps to a problem such as described in this paragraph that inaccuracies in computation and in plotting are liable to accumulate. It is well, therefore, to be somewhat liberal in the amount added to the theoretical lengths in obtaining the practical lengths to be used.

1. **Illustrative Problem** — *Variable Concentrated Loads.* — Find the lengths of the cover plates of the girder designed on page 225: 1. The maximum total bending moment at the center was found on that page to be 3913 thousand pound-feet, including 185 thousand due to the weight of the girder. The concentrated loads were placed as shown in Fig. 251 (a). The corresponding bending moment at the right-hand quarter point (which is greater than at the left-hand quarter point) is $2769 = 175.4 \times 15 + \frac{410}{2} \times 15 \times 45$. The full line of Fig. 262 is plotted from these bending moments. With the loads placed for the maximum bending moment at the quarter point, as in Fig. 251 (b) (see also page 261: 2), the bending moment at the center is $3668 = 194.7 \times 30 - 157.2 \times 15 + 185$, and the bending moment at the quarter point is $3059 = 194.7 \times 15 + 13$ the 138 being the bending moment due to the weight of the girder foun above. The dashed line is plotted from these bending moments. On th diagonal line are laid off the total net area 40.6 sq. in., the net area (the angles and ⅛ of the web $17.8 = 13.9 + 3.9$, the net area of the $14 \times \frac{1}{2}$ plate 8.2, and the net areas of the two $14 \times \frac{5}{8}$ plates each 7.5. The tot area 40.6 is swung about B as a center until it intersects a horizontal lin through C. In this way the parallel lines become horizontal and coinc dent with the horizontal construction lines which cut the curves, whic may, therefore, be drawn directly from the points on the diagonal lin

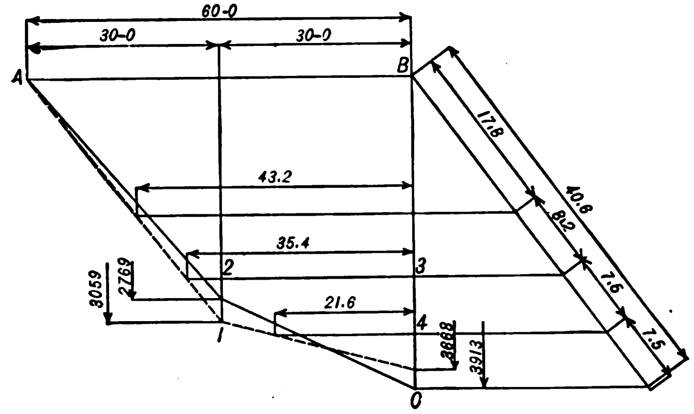


Fig. 262.

to the curve farthest from the center line. In this case the lengths c all three cover plates are determined by the dashed line. Since the di tance AB was made the full length of the girder, the scaled lengths c the cover plates represent the full theoretical lengths. To these shoul be added about three feet to give the actual lengths to be used, thus 46′ 0″ = 43.2 + 2.8, 38′ 6″ = 35.4 + 3.1, and 24′ 6″ = 21.6 + 2.9. On th top flange the $14 \times \frac{11}{16}$ plate should extend the full length of the girde instead of 46′ 0″.

2. For a system of **moving concentrated loads,** such as Cooper's cor ventional engine loads, it is usually impractical to construct an accurat curve of bending moments because this would necessitate finding th

maximum bending moments at short intervals, say equal to the depth of the girder. This would involve a different position of the loads for each bending moment. The resulting curve would approximate a parabola, and it is usually sufficiently close to consider it a parabola, provided a liberal amount is added to the length of each plate to overcome the discrepancy between the curves. Usually from 3 to 4 feet should be added to the theoretical length instead of from 2 to 3, particularly for the shortest plate when more than one are used.

1. An **algebraic method** is more convenient than the graphic method when the curve of bending moments is a parabola. No general algebraic method can be given for girders with fixed concentrated loads because the conditions are so varied. It is possible to compute algebraically the lengths of the cover plates for any specific case by a method adapted from the corresponding graphic method outlined in the preceding paragraphs, but this is not usually recommended for fixed concentrated loads. For either static or moving uniformly distributed loads the curve of maximum bending moments is a parabola, and for moving concentrated loads the curve approximates a parabola, as explained in the preceding paragraph. The curve of net flange areas is also a parabola since the net areas are proportional to the bending moments. The equation of a parabola referred to the origin at the vertex is $X^2 = 4PY$, which shows that values of Y vary as the squares of the corresponding values of X, P being a constant. Hence, $\frac{X^2}{X_1^2} = \frac{Y}{Y_1}$ or $X = X_1\sqrt{\frac{Y}{Y_1}}$. Let us assume that the parabola in Fig. 260 represents net areas instead of bending moments, and that the line BH coincides with BC. Then BC is the total net area required, and the vertex C is the origin of the coördinates. Point A is the only other point on the curve for which the coördinates are known. Neglecting the signs because they do not affect the result, the coördinates of the point A are $X_1 = \frac{L}{2}$, and $Y_1 = a$, where L = the *total* length of the girder, and a = the total net flange area *required*. If for Y we substitute a'' = the distance from the origin to the horizontal line which determines the length of any plate, then the corresponding value of X will be one-half the theoretical length of that plate, or $X = \frac{L}{2}\sqrt{\frac{a''}{a}}$. On account of the usual excess of the actual net area over the required net area, a'' can be found best from a by subtracting the net area of the flange angles, the portion (if any) of the web considered as flange area, and the net area of cover plates between the angles and the cover plate the length of which is desired. For the plate nearest the angles, a'' is equal to the net area left for cover plates, taken directly from the design of the flange. The a'' for the next plate is found from this value by subtracting the net area of the first plate, etc. By doubling both sides of the equation, we have the total theoretical

$$\text{length of cover plate} = L\sqrt{\frac{a''}{a}},$$

the result being in feet since L is in feet.

2. **Illustrative Problem** — *Uniformly Distributed Loads.* — **Find** the lengths of the cover plates of the girder designed on page 222 : 2. The total net flange area required is $a = 30.5$. The resisting moment of the web was neglected, and the net area of the angles was 15.9. The net area of each $14 \times \frac{5}{8}$ plate is 7.5. The a' for the first plate is 14.6 taken from the design (30.5 − 15.9). The a' for the second plate is 7.1 = 14.6 − 7.5. The theoretical lengths are $41.5 = 60\sqrt{\frac{14.6}{30.5}}$ and $28.9 = 60\sqrt{\frac{7.1}{30.5}}$. The actual lengths would be 43′ 6″, and 31′ 0″ on the bottom flange. The longer plate on the top flange should extend the full length of the girder if exposed to the weather.

RIVETS IN COVER PLATES

3. **Four points** must be considered in determining the spacing of the rivets which fasten the cover plates to the flange angles of a plate girder. These rivets must (1) transmit that part of the total flange stress which is carried by the cover plates, (2) provide for the maximum increase in this flange stress, (3) develop the strength of each cover plate between the end of the plate and the end of the next plate, and (4) conform to the general rules for rivet spacing. The first and third points are satisfied when the second is provided for, and often all requirements are fulfilled when the rivets are spaced according to the usual rules for spacing, particularly those given on pages 69 : 1 (*a*), (*b*), (*d*) and 106 : 2. Some companies advocate placing the rivets in the cover plates opposite the flange rivets

through the web in order to simplify both the drafting and the shop work, but as a rule, the benefits derived do not justify the use of the large number of extra rivets.

1. The rivets in the cover plates must carry that proportion of the **increase in flange stress** which the net area of the cover plates bears to the total net flange area. The total increase in horizontal flange stress per linear inch at any point in the girder is $\frac{V}{d_r}$ (page 244: 1) in which V = the maximum vertical shear for a section at the given point, and d_r = the mean depth of the girder between rivet lines in the vertical legs of the flange angles. The portion of this increase which is carried by the cover plates is $\frac{a_1}{a}$, in which a_1 = the net area of all the cover plates at the given point and a = the total net flange area at the same point including any portion of the web counted as flange area. The maximum pitch of the rivets in the cover plates at any point is found by dividing r' = the value of one rivet in single shear by this increase, or

$$p = \frac{ar'\,d_r}{a_1 V}.$$

This formula should be applied at the theoretical end of each cover plate. The resulting pitches usually exceed 6″, so that no further computations need be made. Should the pitches be considerably less than 6″, pitches should be calculated at every point, within the limits of the cover plates, where the pitches of the flange rivets through the web are determined (preceding chapter, page 241). The pitch in the cover plates will always exceed the corresponding pitch of the rivets through the web, but the excess is proportionately less near the center. If the rivets are to be placed opposite those in the web the pitch need not be computed, and if placed opposite alternate rivets in the web only enough pitches need be calculated to determine where this double pitch is insufficient.

2. Each cover plate should be **developed by rivets** between the end of the plate and the end of the next plate. A plate which is fully developed by rivets should fail before the rivets when tested to destruction. A cover plate in tension is, therefore, developed when there are enough rivets to resist the maximum stress which the plate will carry. This is found by multiplying the net area of the plate by the unit stress in tension. Som designers claim that the plate should be developed beyond the theoretica end of the plate, but this is unnecessary, as shown by the curve of bend ing moments in Fig. 260. From the end of the girder to the point 1 th entire flange stress due to bending moment is resisted by the angles an the web. At any point 3 there is an additional bending moment repre sented by the distance 2–3. This increase is provided for by that par of the net area of the plate represented by the distance D–4, and the whol plate need not be developed until the point 5 is reached. The increase i flange stress is not uniform between these points 1 and 5, but ample rivet will be provided if the pitch does not exceed the amount determined b the formula of the preceding paragraph. Each plate extends one or mor feet beyond the theoretical end, and the rivets in this extra length ar spaced not over four diameters (page 69: 1 (*d*)). This insures the de velopment at any point of at least as much of the plate as is required.

3. **Illustrative Problem** — *Uniformly Distributed Loads.* — Find th spacing of the rivets in the cover plates of the girder designed on pag 222: 2. The theoretical lengths of the cover plates were found on pag 263: 2 to be 41.5 and 28.9. The maximum shears in thousands of pound for sections taken at the ends of the cover plates may be found from th maximum end shear of 196.8 (page 248: 1) by the approximate metho (page 247: 3) to be $140.9 = 196.8 \times \frac{\left(\frac{60 + 41.5}{2}\right)^2}{60^2}$ and $108.0 = 196.8 \times \frac{\left(\frac{60 + 28.9}{2}\right)^2}{60^2}$. The value of one rivet in single shear is $r' = 7.22$, an $d_r = 65.25$. At the end of the first plate $a_1 = 7.5$, and $a = 23.4 = 15.9 + 7.5$, the resisting moment of the web being neglected. Th maximum pitch determined by the method of page 264: 1 is

$$\frac{23.4 \times 7.22 \times 65.25}{7.5 \times 140.9}$$

which exceeds 6″. The pitch at the end of the second plate is

$$\frac{30.9 \times 7.22 \times 65.25}{15.0 \times 108.0},$$

which also exceeds 6″. When the first pitch is so large, as in this case $10\frac{1}{2}$″, it is often unnecessary to find the pitch at the ends of the other plates. At the ends of these plates 3″ spaces would be used for about 1′ 9″ = $1\frac{1}{2} \times 14$″ from the actual ends (page 69:1 (*d*)), and 6″ spaces would be used for the remainder.

1. **Illustrative Problem** — *Concentrated Loads.* — Find the spacing of the rivets in the cover plates of the girder designed on page 225:1. From Fig. 262 it is seen that the first two plates end in the first panel where the shear is nearly constant. The smaller pitch will be found in the second plate because the ratio $\frac{a_1}{a}$ is less. This pitch should be found first for if it exceeds 6″ it will be unnecessary to find the pitch at the end of the longest plate. The maximum shear in thousands of pounds for a section at the end of the second cover plate is $202.0 = 194.7 + \frac{0.41 \times 35.4}{2}$, where 194.7 is the maximum shear due to the concentrated loads (page 250:3) and the second term is a simplified expression equal to the weight of the girder (410#/ft) multiplied by one-half the length of the cover plate, which gives the shear due to the weight of the girder. The value of one rivet in single shear is $r' = 7.22$, and $d_r = 65.25$. $a_1 = 15.7 = 8.2 + 7.5$, and $a = 33.5 = 15.7 + 13.9 + 3.9$. The corresponding pitch is $4\frac{3}{4} = \frac{35.5 \times 7.22 \times 65.25}{15.7 \times 202.0}$. Similarly, the pitch at the end of the first cover plate is found to be over $6'' = \frac{26.0 \times 7.22 \times 65.25}{8.2 \times 203.6}$. The maximum shear for a section at the end of the third cover plate is $88.6 = 164.3 - 80.1 + \frac{0.41 \times 21.6}{2}$ and the corresponding pitch is over $6'' = \frac{41.0 \times 7.22 \times 65.25}{23.2 \times 88.6}$. The pitch at the beginning of the second panel would be greater than at the beginning of the third cover plate because the shear is only slightly greater, and the ratio $\frac{a_1}{a}$ is considerably greater. 3″ spacing would be used from the actual end of each plate for a distance of about 1′ 9″ (see preceding problem), and 6″ throughout the remainder of all the cover plates except that $4\frac{3}{4}$″ must be used for the short distance from the 3″ spacing at the end of the second plate to a point beyond the entire connection of the concentrated load at the quarter point.

CHAPTER XXXIX

WEB STIFFENERS

SYNOPSIS: Stiffening angles are riveted to the web plate of a girder wherever the web plate is not strong enough to resist the shearing stresses. These stiffening angles may serve incidentally as connection angles.

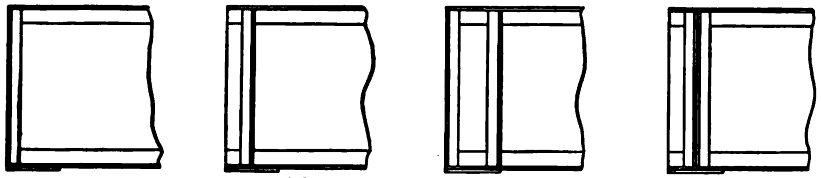
1. The web plate of a girder is designed to resist all **shearing stresses.** The actual course of the diagonal stresses is not known, but the girder can be designed to resist the horizontal and vertical components of the shearing stresses, because these can be determined. The resistance of the web plate to the horizontal components was considered on page 255 : 2. The resistance to the vertical components only need be discussed in this chapter.

2. **Vertical stiffening angles or "stiffeners,"** usually in pairs, are riveted to the web plate of a girder to assist in providing for the vertical shearing stresses when the web plate alone is not sufficient (page 266 : 3). Stiffening angles are also placed at concentrated loads, serving either to connect these loads directly to the web, or else to transmit to the web any loads which rest upon the flange. Stiffening angles are used at each support to transmit the entire reaction, unless the web plate is connected directly to the angles of a column or other member. These stiffening angles may serve as connection angles when the girder is connected to the face of another member, or they may transmit the stress to the bearing when the girder rests upon the support. All stiffening angles should be accurately cut so that they will fit tightly between the outstanding legs of the top and bottom flange angles.

3. **The thickness of the web plate** must be sufficient to satisfy several requirements. It should never be less than $\frac{3}{8}''$ in a railway bridge, nor less than $\frac{5}{16}''$ in a highway bridge or other structure. Many specifications require that the thickness shall be at least $\frac{1}{160}$ of the clear distance between the flange angles, although this is not so general a requirement as some others. It is desirable that any plate over 7 feet deep shall be at least $\frac{7}{16}''$ thick on account of handling in the shop. The area of cross section of the web plate at any point must be sufficient to resist the maximum shear for a section at that point. It is impractical to use more than one web thickness in any girder on account of the method of construction. Web plates are seldom thicker than $\frac{9}{16}''$ or $\frac{5}{8}''$ for it is better to use reinforcing plates where the shear exceeds a certain amount than to use a heavier plate throughout the whole length of the girder. Unless reinforcing plates are used, the web thickness must be such that the product of the gross area of cross section (full depth times thickness) by the corresponding unit shearing stress will equal or exceed the maximum shear on the girder. The gross area is used, and the unit stress is specified for the gross section, because it is not feasible to determine the rivet spacing in the stiffeners before the web thickness is determined, and because the use of net areas would not affect the result appreciably.

4. **End stiffening angles** which connect the web to a supporting member need not be designed as a whole, because the stress increments are transmitted almost directly from the web through the angles to the support, and at no point do the angles have to carry a large cumulative stress. The lengths of the legs are chosen to suit the conditions of each case, usually from 3 to 6 inches. The thickness is usually $\frac{3}{8}''$ for light girders, and $\frac{7}{16}''$ or $\frac{1}{2}''$ for heavier ones. The stiffening angles over the supports of girders which rest upon other members or upon masonry must be designed

to transmit the entire reaction. Since the edge of the web plate is not flush with the backs of the bottom flange angles and can get no direct bearing, the entire maximum shear must be carried from the web plate to the outstanding legs of the flange angles by means of stiffening angles. These angles are sometimes placed at the extreme end of the girder as in (*a*), Fig. 267 (*a*), and sometimes so that the outstanding legs are at the center of the bearing plate, as in (*b*), Fig. 267 (*a*). Angles placed at the extreme end are not so likely to obtain full bearing upon the flange angles, because the latter may be cut short or damaged in cutting. When a girder rests on top of a column, or portion of a column, the outstanding legs of the stiffeners should be placed directly over part of the main column section if practicable. When the reaction is too large to be carried by one pair of stiffeners, two pairs are used, as in (*c*) or (*d*), Fig. 267 (*a*). When the angles



(a) (b) (c) (d)

Fig. 267 (*a*). — **Methods of Placing Stiffeners at the Ends of Girders.**

are placed at opposite ends of the bearing plate, as in (*c*), the pair nearer the center of the girder will get more than half the stress on account of the deflection of the girder. It is difficult to determine the proportion carried by each pair. Sometimes a beveled bearing plate is designed to give an equal distribution under a full load, but this result is difficult to accomplish in practice. More frequently the larger pair is assumed to take two-thirds, and the end angles one-third. This uncertainty is overcome by placing the angles as in (*d*). Sometimes more than two pairs of angles are required, as shown in Fig. 101. The stiffening angles over the support must be designed to transmit the reaction by bearing on the outstanding legs of the flange angles, which in turn transmit the load to the support. The strength of the stiffening angles acting as a compression member must then be investigated.

1. The stiffeners over the support must be **designed for bearing.** The angles are usually riveted against the vertical legs of the flange angles, and fillers are used between the top and bottom flange angles to fill the space between the stiffening angles and the web, as shown in Fig. 97. In order to place the stiffeners in this position the ends must be cut to clear the curved fillet of the flange angles, as shown in the enlarged sketch Fig. 267 (*b*). That part of the stiffeners which is cut back in this way cannot be counted in bearing, because the usual shop methods do not insure contact. Even if perfect contact were obtained, the value of the bearing upon the curved surface of the fillet would be questionable. The fillets of 6×4 and 6×6 flange angles are $\frac{1}{2}''$ in radius, and those of 8×8 angles are $\frac{5}{8}''$. Unless the stiffeners are more than $\frac{1}{16}''$ thicker than the radius of the fillets, the bearing of the web legs (i.e., the legs riveted to the web) must be ignored, and the portion of the outstanding legs beyond the fillets of the flange angles must carry the whole load. The necessary thickness of the angles is found by dividing the maximum reaction by the allowed unit stress in bearing, and again by the combined projections of the outstanding legs of the stiffeners beyond the fillets of the flange angles. If the thickness of the angles exceeds the radius of the fillet by more than $\frac{1}{16}''$ (to allow for inaccurate cutting), the bearing of the remaining portion of the web legs may be counted.

Fig. 267 (*b*).

The unit stress in bearing may be taken the same as the bearing value for pins and shop rivets (24,000#/ sq. in. according to the specifications of the American Railway Engineering Association). Some designers use a smaller unit stress and count both legs of the angles, but it is more logical to count only that portion which is sure to bear. The outstanding legs are less likely to buckle when designed in this manner. Some engineers design the stiffeners as compression members without considering bearing, with the result that the outstanding legs are often seriously overstressed. It is better to design the angles for bearing, and then investigate their strength as a compression member. The lengths of the outstanding legs are usually made of the largest commercial sizes which will not project beyond the edges of the flange angles, but circumstances may justify cutting larger angles so that they will be flush with the edges of the flange angles. The web legs are shorter than the outstanding legs, standard angles being used. The thickness should not be less than $\frac{3}{8}''$, and if more than $\frac{3}{4}''$ the angles must be sub-punched or drilled.

1. **The strength in compression** of the stiffeners over the support should be investigated. The stress in the angles is cumulative throughout the depth of the web, and the full stress acts only at the bottom. It is, therefore, unnecessary to consider the full load acting at the top of the stiffening angles, but it is customary to consider the length of the compression member equal to one-half the depth of the girder. The radius of gyration is chosen for an axis through the center of the web, because the angles are restrained in the other direction by being riveted to the web. Only the area of the angles is considered, the included portion of the web being neglected. The usual unit stress in compression is used, as for example, $16{,}000 - 70\,\frac{l}{r}$.

2. There must be enough **rivets** in each pair of stiffening angles over the supports to transmit the whole stress taken by the angles. The rivets which pass through the flange angles should not be counted upon to take any of this vertical stress, because they are fully developed in transmitting the horizontal stress from the web to the flange angles. The rivets are driven in the shop, and the value of one rivet is limited by the bearing in the web (unless the double-shear value is less).

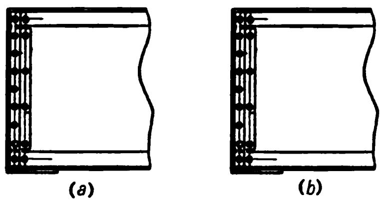


Fig. 268 (*a*).

Rivets which pass through fillers are less effective than those which connect the parts directly, because the rivets are more likely to bend. The usual specifications provide for this by requiring arbitrarily that the number of rivets be increased 50% when they pass through fillers. Whenever feasible it is well to increase the width of the filler so that some or all of the extra rivets may be used to connect the fillers to the web without passing through the angles, as shown in (*a*), Fig. 268 (*a*). When pairs of stiffening angles are close together these fillers may extend under both pairs, as in (*b*), Fig. 268 (*a*). These extra rivets should be spaced not more than twice the maximum distance allowed in the angles, which is sometimes specified 5″ instead of the usual 6″. If the grip of the rivets (i.e., the total thickness of the parts connected) exceeds four times the diameter, the number should be increased 1% for each additional sixteenth of an inch in the grip.

3. **Illustrative Problem.**— *Angles at Edges of Bearing Plate.* Design the end stiffening angles of the girder shown on page 222:2, arranging them as in Fig. 268 (*b*). The outstanding legs of the flange angles are 6″, so we will use $5 \times 3\frac{1}{2}$ stiffeners. The maximum end shear was found on page 248:1 to be 196,800#, and two-thirds of this will be assumed to be carried by the inner pair of angles. If the angles are not thicker than the $\frac{1}{2}$″ fillet, the length of bearing will be $9'' = 2\,(5 - \frac{1}{2})$, and the corresponding thickness must be $\frac{5}{8}'' = \frac{2 \times 196{,}800}{3 \times 9 \times 24{,}000}$. This is $\frac{1}{8}$″ more than the radius of the fillet, but if smaller angles were used they would not exceed the radius by more than $\frac{1}{16}$″ which is negligible, so $\frac{5}{8}$″ angles will be used. The two angles are separated by the $\frac{7}{16}$″ web and two $\frac{7}{8}$″ flange angles. The half depth of the girder is 3 ft. From the table on page 331, the safe load of 2 Ls $5 \times 3\frac{1}{2} \times \frac{5}{8} \times 3'\,0''$ is 132,000 when the least radius of gyration about an axis perpendicular to the web is used. A larger safe load determined by the radius about the other axis could be used, because the angles are restrained, but this value need not be found in this case because 132,000 is equal to the necessary two-thirds of 196,800. The number of $\frac{7}{8}$″ rivets bearing in the $\frac{7}{16}$″ web plate, required in these stiffening angles is $15 = \frac{2 \times 196{,}800}{3 \times 9190}$, and the total number in the angles and the filler is $22 = \frac{2 \times 196{,}800 \times 1.5}{3 \times 9190}$. Thus 15 rivets should be used in the angles * exclusive of the two in the flange angles, and 7 additional rivets should be used in the fillers. The angles at the end of the girder carry only one-half the load of the angles just designed, so they need be only one-half as thick, the $\frac{3}{8}$″ minimum being sufficient. The number of rivets should be $11 = 22 \div 2$, but four more are used so that the rivets line up with those in the other angles.

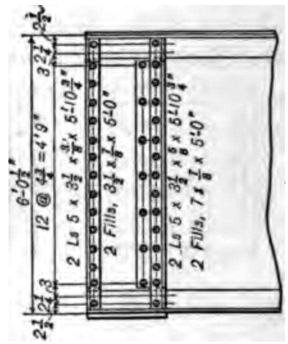


Fig. 268 (*b*).

* Two more than shown in Fig. 268 (*b*).

1. **Illustrative Problem** — *Angles at Center of Bearing Plate.* — Design the end stiffening angles of the girder designed on page 225: 1. The maximum end shear was found on page 250: 3 to be 207,000#. The effective length of bearing in two $5 \times 3\frac{1}{2}$ angles is $9 = 2(5 - \frac{1}{2})$, and the corresponding thickness would have to be $1'' = \dfrac{207{,}000}{9 \times 24{,}000}$. Since no such angles are rolled, and since they would have to be drilled if available, four angles will be used, each $\frac{1}{2}''$ thick, arranged as in Fig. 269. The safe load would be considerably greater than that shown in the table on page 331 (see preceding problem), but for two pairs of angles this is $2 \times 108{,}000$ so that no further investigation is required. The number of $\frac{7}{8}''$ rivets in each pair of angles, increased 50% on account of the fillers, is $17^* = \dfrac{207{,}000 \times 1.5}{2 \times 9190}$, exclusive of those through the flange angles. These are all placed through the angles, and spaced to line up with the rivets in the splice plates, Fig. 273. Seven-inch fillers are used extending under both pairs of angles. If wider fillers were used, the total number of rivets would be increased or else the maximum spacing would be exceeded.

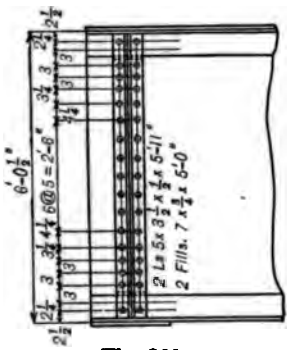


Fig. 269.

2. **Intermediate stiffeners** are generally used at concentrated loads and at web splices. They are also used in the deeper girders to prevent the web from buckling under its shearing stresses. The more modern specifications require the use of intermediate stiffeners when the thickness of the web is less than $\frac{1}{60}$ of the unsupported clear distance between the flange angles. Formerly some form of "buckle formula" (page 293: 3) was used to determine when stiffeners should be used. The sizes of intermediate stiffening angles are not designed, but they are usually fixed by the standards of the different structural companies. The specifications often require that the length of the outstanding legs should be at least 2" greater than $\frac{1}{30}$ of the nominal depth of the girder. The intermediate stiffeners are usually made lighter than the end stiffeners, and often the legs are of the next smaller commercial size, excepting perhaps those which serve as connection angles for concentrated loads. The thickness of intermediate stiffening angles in railway bridges is usually $\frac{3}{8}''$, although this may be increased in the heavier girders. The thickness of the intermediate stiffening angles in other girders is usually either $\frac{5}{16}''$ or $\frac{3}{8}''$.

* Two more than shown in the figure.

3. **The spacing of intermediate stiffeners** should not exceed the maximum allowed by different clauses in the specifications. The stiffeners at concentrated loads are usually fixed in position. In railway bridges, viaducts, and heavy crane runways, the clear distance between stiffeners should never be more than 6 feet, nor more than the clear distance between the flange angles; it should not exceed $\dfrac{t}{40}(12{,}000 - v)$, in which t = the thickness, and v = the maximum shear intensity, in pounds per square inch of gross section. This formula provides for closer spacing near the ends of the girder where the shear intensity is greater. In highway bridges and structures other than those mentioned above, the clear distance between stiffeners should not exceed 5 feet, nor the full depth of the web.

4. The number of **rivets** in intermediate stiffeners is not computed, but the rivets are made to line up with those in the stiffeners at the supports. This simplifies both the drafting and the shopwork, particularly when multiple plate punches are used. The full number is used in the stiffeners at concentrated loads and at splices, but in other stiffeners the alternate rivets may be omitted if the resulting spaces do not exceed the allowed maximum. Care should be taken to see that the number of rivets in stiffeners used as connection angles fully provide for the loads.

5. Intermediate stiffening angles which have no connections to the outstanding legs may be **crimped,** as explained on page 97: 1.

CHAPTER XL

SPLICES

SYNOPSIS: The design of typical splices is illustrated by different types of girder and column splices.

1. **Definition.** — One member is usually connected to another by means of connection plates or angles, but a member or part of a member is said to be spliced when the two similar segments are connected end to end in the form of a butt joint. A splice becomes necessary when the desired length of material is not available, or when it is impracticable to ship a whole member in one piece. Whenever possible a splice should be avoided.

2. **Design.** — A splice should be designed to transmit the full stress in the part spliced. In a tension member or a light compression member the splice plates or angles should be designed to carry the full stress. This is not feasible in a heavy column or chord member, so part of the stress is transmitted by direct bearing, the ends of the two segments being "milled" or "faced" (page 31: 1) to furnish uniform bearing against each other, as shown in Figs. 124 and 133. The rivets in each half of the splice plates or angles should fully develop that part of the stress which is carried by the corresponding plates or angles. Thus in a tension member, the combined net areas of the splice plates or angles should equal or exceed the net area for which the main member is designed, as illustrated by the problems on page 208: 4, and the rivets which connect the splice plates or angles to *each segment* should develop the full strength of the main member. In a milled splice of a heavy compression member most of the stress is transmitted by direct bearing, but enough splice plates or angles should be used to hold the segments in the proper position and to transmit the stresses due to bending.

3. **Three types** of splice are selected to illustrate the principles embodied in the design of any ordinary splice. These three will be discussed in the following order: a splice in the web of a plate girder, a splice in the flange of a plate girder, and a column splice.

WEB SPLICES

4. **When Used.** — The web plates of all girders over 40 or 50 feet long must be spliced because longer plates are not rolled. The extreme lengths of some of the deeper webs are less than 20 feet so that the webs of the longer girders must be spliced at several points. The number of splices is determined by the maximum rolled lengths * of the plates, but for convenience in handling the plates in the shop no single plate should weigh more than 3000#. A splice should not be located at the point of maximum bending moment, except possibly in comparatively light girders which would then require only one splice. It is usually customary to locate each splice under a pair of stiffening angles. Not only is the splice thus stiffened, but the thickness of the fillers is reduced, and there is one less line of rivets to be driven.

5. **The design of a web splice** depends upon the method by which the girder is designed, because the splice plates must transmit the web stresses. If the entire flange stress is considered to be resisted by the flange, and the resisting moment of the web plate is neglected (as in Case A, page 221: 2), the splice is designed for shearing stresses only. If the resisting moment of the web isconsidered (as in Case B, page 223: 2), the splice must be designed for stresses due both to shear and to bending

* See Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York, or the handbooks of steel manufacturers.

moment. When there are more than two splices in a girder, the one for which the shear is greatest is designed and the others are made like it for practical advantages during fabrication. Splice plates are placed on both sides of the web, and the thickness of each plate should not be less than $\frac{3}{8}''$ in railway bridges or $\frac{5}{16}''$ in other structures.

1. **When the splice is designed for shear only** the plates extend the full depth between flange angles, except for a clearance of $\frac{1}{4}''$ (or less) at each end, being the same length as the fillers (page 96 : 4). The net area of the two splice plates in a vertical cross section should be as great as the net area of the web plate, but for all practical purposes it is close enough to compare the gross areas since the proportions are so nearly identical. The width of the splice plates is determined by the spacing of the rows of rivets which often depends upon the size of the superimposed stiffening angles. A space of from $\frac{1}{4}''$ to $\frac{3}{4}''$ is usually left between the two sections of the web (page 96: 2). At least two rows of rivets are commonly used in each half, even though one row would satisfy the conditions. Usually the rivets in the inner rows are spaced like those in the intermediate stiffening angles (page 269 : 4), but those in the outer rows are spaced twice as far apart. After the rivets have been located in this manner their resistance to shear should be investigated. The shear, or the algebraic sum of the vertical external forces on one side of the splice, is positive, while the shear on the other side is of equal magnitude but negative, in order to satisfy the V equation of equilibrium. Since the shearing stresses are transferred from one segment of the web to the other by means of the splice plates and the rivets, the effect is the same as if the maximum shear V were a single resultant force acting upward at the center of gravity of the group of rivets in one-half the splice, with an equal downward resultant force acting at the center of gravity of the rivets in the other half. A couple is thus formed, the moment of which is the product of a single force V by the perpendicular distance between the two forces. This moment tends to cause rotation of the splice plates, and the splice is a double eccentric connection. The rivets in each half of the splice must not only satisfy the maximum vertical shear V, but they must resist one-half of this moment according to the method of eccentric connections, page 237 : 2. Should the strength of the rivets prove to be insufficient, the number in the outer row may be increased by one or more, the number in the inner row may be made the same as the rivets in the end stiffeners if not already the same, the rows may be spread slightly, or the number of rows may be increased.

2. **Illustrative Problem** — *Web Splice for Shear Only.* — Design a splice for the web at the quarter point of the girder shown on page 222 : 2. The maximum shear for a section at the quarter point in thousands of pounds may be found from the maximum end shear by the approximate method (page 247: 3) to be $110.7 = 196.8 \times (\frac{3}{4})^2$. The length of the splice plates is $60'' = 72.5 - 2(6 + \frac{1}{4})$, and the thickness of each plate is found by equating the gross area of two splice plates to the gross area of the web plate thus: $2 \times 60t = 72 \times \frac{7}{16}$; this gives a value for t less than the minimum, so $\frac{3}{8}''$ plates are used. The minimum number of rivets is as shown in Fig. 271, the spacing in the intermediate stiffeners being the same as in the end angles shown in Fig. 268 (*b*) because the double pitch would exceed the maximum allowed. Since the inner row has approximately twice as many rivets as the outer row, the center of gravity may be taken one-third of the distance from the inner row to the outer row. The distance from this center of gravity to the center of the splice is $3'' = 1 + 2$, which is half the distance between the centers of gravity of the two groups of rivets.

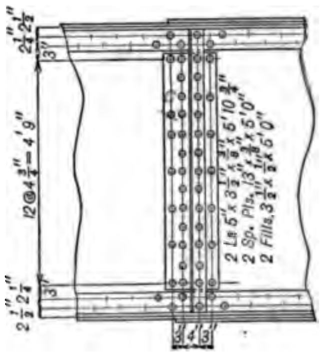


Fig. 271.

The half moment of the couple is then $332{,}100\#/\text{ in.} = 110{,}700 \times 3$. The sum of the squares of the horizontal and vertical distances to the rivets is $6670 = 7 \times 2^2 + 13 \times 1^2 + 4(9\frac{1}{2}^2 + 19^2 + 28\frac{1}{2}^2) + 2(4\frac{3}{4}^2 + 14\frac{1}{4}^2 + 23\frac{3}{4}^2)$. The coördinates of the rivet farthest from the center of rotation are $x_m = 1$, and $y_m = 28\frac{1}{2}$. The horizontal component of the shearing forces on the critical rivet is $1420\# = \frac{332{,}100 \times 28.5}{6670}$, and the vertical component is $5850\# = \frac{110{,}700}{20} + \frac{332{,}100 \times 1}{6670}$. The resultant of these two components is obviously less than 9190#, the value of a $\frac{7}{8}''$ rivet bearing in a $\frac{7}{16}''$ web, so no more rivets need be added.

1. **When the splice is designed for bending moment as well as for shear** the same plates may serve to resist both, or separate plates near the flange may be used to resist the bending moment, as explained in the next paragraph. The portion of the bending moment which is resisted by the web may vary somewhat on account of the abrupt change in flange area at the ends of the cover plates, but the splice should be designed to transmit the full resisting moment of the web plate. This resisting moment at any point in the girder is found by multiplying $\frac{1}{8}$ of the gross area of the web plate (or other portion used in designing the flange) by the unit stress in bending and by the effective depth of the girder from center to center of gravity of the flanges (page 224: 1). When only one pair of splice plates is used, their section modulus as well as the area of cross section of the plates should equal or exceed that of the web plate. Thus, the square of the depth of the splice plates multiplied by their combined thickness should not be less than the square of the depth of the web plate multiplied by the web thickness. The strength of the rivets in each half of the plates should be investigated in the same manner as those in a splice designed for shear only (see above) except that they must resist not only the vertical shear and the half moment due to shear, but they must also resist the full resisting moment of the web as described in this paragraph. Although the full resisting moment of the web is not always developed at the same time that the maximum shear exists, it is customary to design the splice as if the two maximums occurred simultaneously. This type of splice is sometimes rendered more effective by spacing the rivets closer together near the flanges than near the neutral axis. When this is done, the rivets in all the stiffening angles should be spaced accordingly.

2. **Separate splice plates** may be designed to resist that part of the bending moment carried by the web when the single pair of plates described in the preceding paragraph prove impractical. Sometimes these additional plates are placed against the vertical legs of the flange angles,* but more frequently the central plates are cut short and the additional plates are placed between them and the angles, as shown in Fig. 272. These additional plates are designed to resist the bending moment, and the central plates are designed to resist the shear. The former must be designed first because their width determines the length of the central plates. Usually a clearance of $\frac{1}{4}''$ (or less, page 96 : 4) is left between the shear and the moment plates, and between the moment plates and the flange angles. The resisting moment of the two sets of moment plates must equal or exceed the resisting moment of the web plate found as explained in the preceding paragraph. The distance from center to center of splice plates d_s is considerably less than the effective depth of the girder d_g, as shown in Fig. 272. For the same unit stress, the product of the net area of one pair of splice plates by the distance d_s must equal the product of $\frac{1}{8}$ the gross area of the web by the depth d_g, or the *net* area of one pair of splice plates must be at least $\frac{d_g}{d_s}$ times the $\frac{1}{8}$ (or other portion) of the *gross* area of the web considered as flange area. The width of the splice plates depends upon the number and the spacing of the rows of rivets. A trial design may be made with 8″ or 9″ plates allowing for three rows of rivets spaced $2\frac{1}{2}''$ or 3″ apart. The combined thickness of the moment plates is found by dividing the net area by the net width. No plate should be thicker than the flange angles. The number of rivets in each half should be sufficient to fully develop the tensile strength of the plates selected, at the same unit stress for which the flange was designed. The length of the plates depends upon the number and the spacing of the rivets. The rivets may be placed opposite (thus making the plates shorter) or staggered (making the plates thinner), as best suits the superimposed stiffeners. The proportions may be changed if desired by changing the number of rows of rivets. After the moment plates are determined, the central shear plates may be designed in the same manner as on page 271 : 1, the gross area of the two plates being as large as the gross area of the whole web, and the strength of the rivets being sufficient to resist both the direct shear and the resulting moment due to eccentricity.

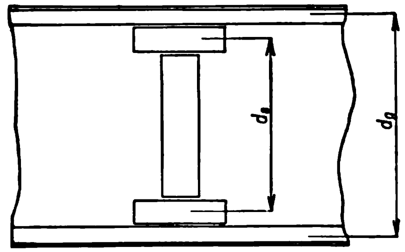


Fig. 272.

* See Kunz's "Design of Steel Bridges," McGraw-Hill Book Co., Inc., New York, or Johnson-Bryan-Turneaure's "Modern Framed Structures," Part III, John Wiley and Sons, Inc., New York.

1. **Illustrative Problem** — *Web Splices for Moment and Shear.* — Design a splice for the girder shown on page 225 : 1, assuming the web to be cut into three lengths of about 20 feet. Assume 9″ moment plates with three rows of rivets, as shown in Fig. 273.

$72.3'' =$ effective depth d_e (page 225 : 1)

$51.0'' = 72\frac{1}{2} - 2(6 + \frac{1}{4}) - 9 = d_s =$ distance c. to c. of splices

3.9 sq. in. $= \frac{1}{8}$ of the gross area of the web (page 225 : 1)

$$\tfrac{7}{16}'' = \frac{72.3 \times 3.9}{51.0 \times 2\,(9 - 3 \times 1)} = \text{thickness of each moment plate}$$

$84{,}000\# = 2 \times 6 \times \frac{7}{16} \times 16{,}000 =$ developed strength of two plates

$10 =$ number of $\frac{7}{8}''$ rivets bearing in $\frac{7}{16}''$ web (page 310).

These rivets may be satisfactorily arranged as in the figure, so no change need be made in the number of rows.

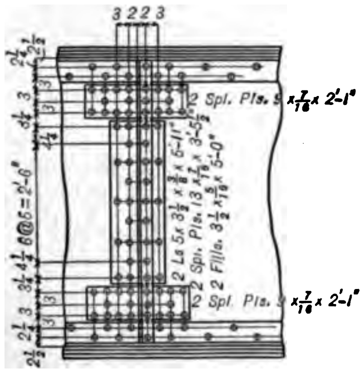


Fig. 273.

$41.5'' = 51.0 - 9 - 2 \times \frac{1}{4} =$ depth of the central shear plates

$$\tfrac{3}{8}'' = \frac{72 \times \frac{7}{16}}{2 \times 41.5} = \text{required thickness}$$

of each shear plate. This is so nearly equal to the thickness of the moment plates that it would be increased to $\frac{7}{16}''$ so that the fillers under the stiffening angles may extend over all the splice plates. The vertical spacing of the rivets is arranged as shown to accommodate both the splices and the stiffening angles (page 269 : 1). Alternate rivets are omitted in the outer rows of the central plates, and the strength of the remaining rivets is investigated as on page 271 : 2. The distance from the center of gravity of the rivets to the center of the splice is 3″ as in the preceding problem.

$88{,}300\# = 90{,}400 - 410 \times 5 =$ maximum shear (page 250 : 3)

$2410 \quad = 5 \times 2^2 + 9 \times 1^2 + 4(10^2 + 19\frac{1}{4}^2) = 2(5^2 + 15^2) =$ sum of squares

$$2120\# = \frac{88{,}300 \times 3 \times 19.25}{2410} = \text{hor. comp. on critical rivet}$$

$$6420\# = \frac{88{,}300}{14} + \frac{88{,}300 \times 3 \times 1}{2410} = \text{vert. comp. on critical rivet}$$

The resultant of these components is obviously less than 9190#, the value of a $\frac{7}{8}''$ rivet bearing in a $\frac{7}{16}''$ web, so the splice is satisfactory.

FLANGE SPLICES

2. **When Used.** — Flange splices should be avoided wherever possible. Cover plates more than 80 feet long must be spliced; 6×6 flange angles are rolled in some mills * up to 100 feet in length, and 8×8 angles up to 120 feet. Special arrangements may be made for rolling longer angles, and it is often better to pay the extra cost than to splice the angles. When bridge girders are built with curved ends, as in Fig. 101, it is impractical to bend the long flange angles or cover plates and they should be spliced near the curves. Most flange splices are made in the shop, because most girders are shipped complete as single members. Girders for export, or girders which have to be hauled through crowded streets, may have to be shipped in sections and joined at the site in order to meet the shipping requirements.

3. No two **component parts** of a girder should be spliced at the same point (except as provided on page 274 : 3) and preferably not within 2 feet of each other so that the stresses can properly readjust themselves between splices. It is usually permissible to splice a top angle vertically over a bottom angle provided they are on opposite sides of the web. In order to keep the spacing symmetrical, the remaining angles are usually spliced at an equal distance the other side of the center line. Angle splices should preferably be placed where there is an excess in flange area, i.e., near the end of a cover plate which is not used as a splice plate. Flange splices are usually made without clearance between the ends of the spliced plates and angles for the sake of stiffness, but the ends cannot be milled after assembling and it is difficult to obtain perfect bearing. The splice of a compression flange is therefore made like a

* See Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

splice for a tension flange, no allowance being made for the direct bearing of the abutting ends.

1. **A cover plate** is spliced usually at the point where another cover-plate would otherwise end, so that the latter may be extended to serve as a splice plate. In this way less material and fewer rivets are required than if a separate splice plate were used. At this point of splice one continuous plate is cut, and another plate begins. In effect this means that the stress in the single plate on one side of the splice must be transmitted to one of the two plates on the other side, but to which one is immaterial provided the net area is the same. It simplifies the analysis to consider that the stress is imparted to the outer plate, although the actual distribution of the stress between the two plates does not affect the result. The outer plate may be extended beyond the splice to form a lap joint with the single plate, enough rivets being used, acting in a single shear, to fully develop the plate. The rivets are usually spaced about 3″ apart to reduce the length of the outer plate. The splice is thus completed on one side of the original point of splice. The remaining section of the cut plate is considered to act as the new plate which begins at the point of splice, and the rivets should be spaced close together for the usual distance from this end (page 69 : 1 (*d*)).

2. **A flange angle** is spliced by means of a "cover angle" or "splice angle" the vertex of which must be cut to fit the curved fillet of the flange angle, as shown in Fig. 274. The legs of these angles are cut so the edges will be approximately flush with the nominal edges of the flange angles. Special gages are usually required to give the necessary driving clearance. If possible, the net area of the splice angle should be as large as the net area of the flange angle, otherwise the deficiency must be made up by a plate riveted to the opposite flange angle, as shown in the figure.

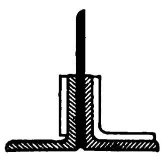


Fig. 274.

The net area of the splice angle is the product of the thickness by the value found by subtracting the thickness of the angle and the diameter of each hole in any one crosssection (usually two, page 221 : 2) from the sum of the two cut legs. No consideration need be made of the part of the angle which is cut away to clear the fillet of the flange angle because this is compensated for by the fillet of the splice angle itself. The rivets in each half of the splice angle must fully develop the splice angle. The same flange rivets which are used to transmit the flange stress from the web to the flange angles may be counted for this purpose because the shearing plane is on the opposite side of the flange angle. The stress developed in each rivet between the web and the flange angles depends upon the double shear value or the bearing value in the web, whichever is smaller. One-half of this value represents the stress in the plane between the web and one flange angle. This amount should be subtracted from the bearing value in the flange angle to show the bearing value available in the plane between the flange angle and the splice angle. This difference should be compared with the single shear value, and the smaller taken as the limiting value of each rivet through the web leg. Usually the flange angles are so thick that the full single shear value may be used. This is especially true when extra rivets are used, i.e., when the spacing is less than the pitch determined by the flange stress. The limiting value of each rivet through the outstanding leg will be the single-shear value, even though cover plates are connected by the same rivets. If a splice plate is used on the opposite side of the flange to supplement the splice angle, the number of rivets in the plate is determined independently. If the plate were in contact with the angle to be spliced, the value of each rivet would be the same as that in the web leg of the splice angle, and a corresponding number of rivets should be found which would fully develop the splice plate. However, this number must be increased by 33⅓% for each intervening plate according to the usual specification for indirect splices. If there are no vertical flange plates, the web and the continuous flange angle make two intervening "plates." Splices are usually made as short as practical by spacing the rivets 3″ apart, or less, a due consideration being given to the minimum spacing allowed (page 255 : 2). Flange splices should logically be placed in a panel between stiffening angles so they will not interfere with the stiffeners.

3. **The splice at the curved end** of the top flange of a girder occurs where there is a large excess of flange area, especially when the cover plate extends to the end. It is therefore permissible to cut both flange angles at the same point in this case, and it is usually unnecessary to fully develop the angles or the plate, provided the splices are designed to safely carry the maximum stresses which can reach them. Such

splices are shown in Fig. 101. When the whole reaction is carried by stiffening angles placed at the center of the bearing, end angles may be used with a small curve at the top which extends only to the main stiffeners, meeting the main flange angles at that point, as shown in Fig. 21. No splice is necessary, and smaller angles may be used since their principal function is to improve the appearance of the end of the girder. The end cover plate should be of the same width as the top plate.

1. When a **field splice** becomes necessary it should be made as compact as possible in order to reduce the number of field rivets and the lengths of the different sections of the girder. Some specifications require that no two splices in a flange shall be within two feet of each other but this requirement would make a field splice much longer than seems to be necessary. A typical flange splice is shown in Fig. 275. The web is spliced as explained in the first part of this chapter. One angle is spliced to the left of the web splice and the other to the right, two splice angles of equal length being used for both splices. For convenience, the rivets in the two legs are placed opposite. One cover plate is spliced at the left end of the splice angles and the other at the right end, the splice nearer the center of the girder being located, if possible, at a point where the third cover plate may be extended to serve as a splice plate. The net area of each splice angle is usually about three-fourths of the net area of one flange angle. It is considered good practice, in order to stiffen the whole splice, to place enough rivets through the vertical legs of the flange angles between the two splices to develop one flange angle. Thus a lap joint is formed which transmits the stress of one flange angle, and the splice angles need carry only the stress of the other flange angle. No excess need be added on account of the web intervening, because the number of rivets determined by the single-shear value together with the rivets in the outstanding legs will be more than would be required for two independent splices. The rivets in both legs of the splice angles beyond each splice is determined by the number in single shear required to develop the splice angle next to the flange angle which is spliced. At least as many of these rivets should be placed in the web leg as are required to transmit the remaining stress to the opposite splice angle, allowing two-thirds excess on account of the intervening web and flange angle. The splice plate should extend beyond the last cover-plate splice to include enough rivets acting in single shear to develop one plate. It would be preferable to splice the outer plate at this point so that it would be in contact with the splice plate, but it is not customary to add an excess even though another plate intervenes, as in the figure, because all plates act together. No special consideration need be given to the remaining rivets in the cover plates because there will be an excess due to the fact that they are placed opposite those in the web legs of the angles. Close spacing should be continued beyond the inner splice because an additional plate begins at this point (page 274 : 1).

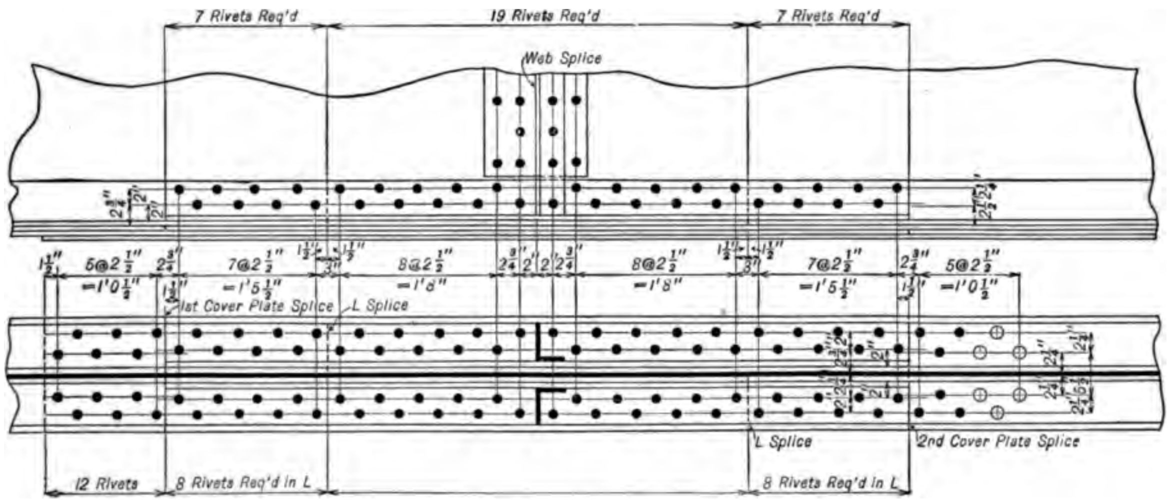
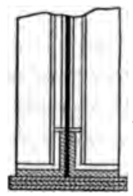


Fig. 275. Typical Field Splice in a Girder Flange.



1. **Illustrative Problem** — *Field Flange Splice.* — A plate girder is to be spliced in the field. Each flange is composed of 2 Ls $6 \times 6 \times \frac{3}{4}$, and three cover plates $14 \times \frac{3}{8}$, designed at 16,000#/sq. in. Design the flange splice, using $\frac{7}{8}''$ rivets.

6.94 sq. in. $= 8.44 - 2 \times 1 \times \frac{3}{4}$ = net area of one flange angle
5.21 sq. in. $= 6.94 \times \frac{3}{4}$ = net area required in each splice angle
5.37 sq. in. $= (2 \times 5\frac{1}{4} - \frac{11}{16} - 2 \times 1)\frac{11}{16}$ = actual net area of 1 L $5\frac{1}{4} \times 5\frac{1}{4} \times \frac{11}{16}$
111,000# $= 6.94 \times 16{,}000$ = developed stress of one flange angle
85,900# $= 5.37 \times 16{,}000$ = " " " " splice "
41,800# $= (111{,}000 - 85{,}900)\frac{5}{3}$ = balance of stress in opposite splice angle with two-thirds excess
72,000# $= (14 - 2 \times 1)\frac{3}{8} \times 16{,}000$ = developed stress of one cover plate.

From the upper table on page 310 the value of one field rivet in single shear is 6000#. The number of rivets required in the vertical legs of the flange angles between splices is $19 = 111{,}000 \div 6000$. The total number required in each splice angle beyond the splice is $15 = 85{,}900 \div 6{,}000$, of which $7 = 41{,}800 \div 6000$ must be in the web leg. The number in the cover plate beyond the last splice is $12 = 72{,}000 \div 6000$. The completed splice is shown in Fig. 275.

COLUMN SPLICES

2. The splices for **mill-building columns** are so simple that they may be designed without difficulty. They are only used, as a rule, where the width of the column is changed to accommodate a crane girder (Fig. 135), or where a gusset plate replaces part of the web. These splices are comparatively light and enough rivets are used to fully develop the compressive strength of the part spliced.

3. **Office-building columns** are usually built in two-story lengths and many splices must be used in the larger structures. For convenience in erection these splices are located just above the connections of the floor beams and girders. The heavy cumulative loads make the use of milled ends imperative, and usually most of the stress is transmitted by direct bearing. When the full area of cross section of the upper column bears upon the section immediately below it, splice plates are used simply to hold the columns in the proper relative position and to transmit the bending stresses. These plates are usually more or less standardized by structural companies for different types of columns so that they need not be designed for each column. At some points in tall office buildings it is necessary to make a decided change in the dimensions so that only a small portion of the upper section bears directly upon the lower section, and special splices must be designed. Only one common form will be discussed here, although similar splices can be adapted to different types of columns.

4. **Channel columns** are composed of two channels with either lattice bars or cover plates across their flanges, as shown in Fig. 133. The top sections are usually latticed; in the lower sections, as the column loads increase, the lattice bars are replaced by cover plates, and the plates and the channels are made heavier, until a point is reached where deeper channels and wider plates become necessary. Splice plates are used on the flanges, and light splice angles are used on the webs, as shown at the top of *AB*1 or at the bottom of *EF*27, in the figure referred to. When the channel depth and plate width are changed the conditions are as shown in Fig. 277 (*b*). Provision must be made for transmitting most of the load by other means since only a small portion of one column bears upon the other. Reinforcing plates are used to properly distribute the bearing, supplemented by a $\frac{1}{2}''$ or $\frac{3}{4}''$ bearing plate which is placed between the ends of the two columns. The stress in the upper cover plates is transferred to the lower cover plates by reinforcing plates which are riveted to the upper cover plates as shown in Fig. 277 (*a*). The combined thickness of these reinforcing plates or fillers on each side of the column is made $\frac{1}{16}''$ less than the horizontal distance from the outside of the upper cover plate to the outside of the lower cover plate in order to give a total clearance of $\frac{1}{8}''$ to facilitate the erection of the upper column between the splice plates which are riveted to the lower column. The reinforcing plates should be attached to the upper column by enough rivets in single shear to develop the cover plate. The field rivets which connect the splice plate may be counted upon to transmit their full share of the stress from the cover plate to the reinforcing plates because the two shearing surfaces are so far apart. Due attention should be paid to the value of countersunk rivets (page 231: 2) but usually the thickness of the plates is sufficient to justify the use of the full single-shear value. The stress in the upper channels is transferred to the lower channels in a similar manner by reinforcing plates riveted to the lower channels to

form shelves. The combined thickness of these reinforcing plates on each channel is made from $\frac{1}{16}''$ to $\frac{1}{8}''$ less than the horizontal distance from the face of the upper web to the face of the lower web, this reduction

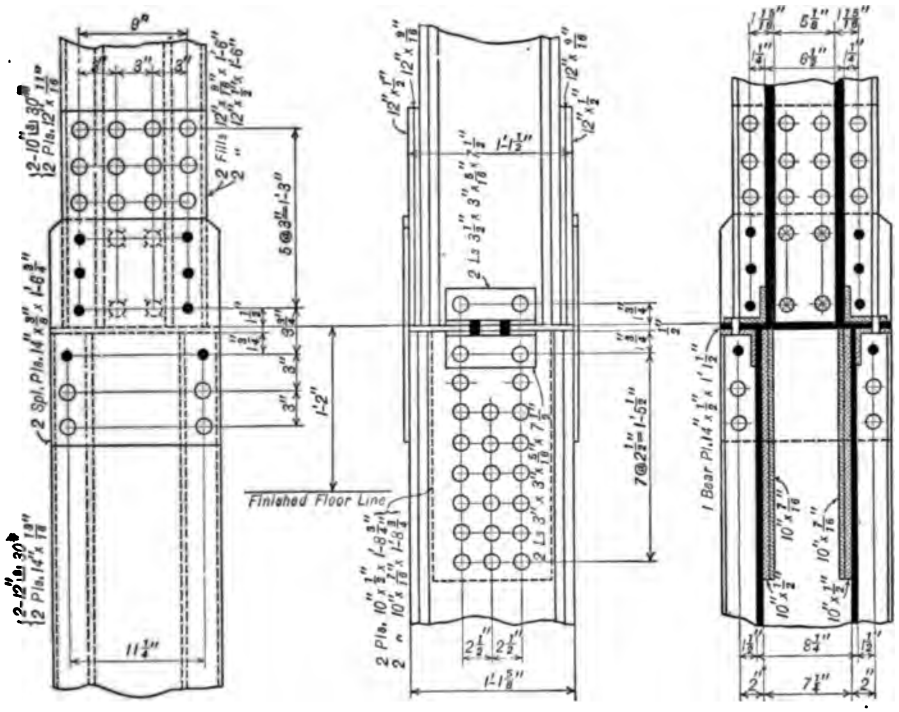


Fig. 277 (*a*). Channel Column Splice.

being justified by the use of the bearing plate which distributes the stresses. The width of these reinforcing plates is made equal to the depth of the upper channel, and the number of rivets should develop the stress in this channel. In developing the cover plates and the channels it is desirable to use the unit stress for which the column was designed. If this is not available, the unit stress should be found from the proper compression formula, substituting the proper length and radius of gyration.

When the formula $16{,}000 - 70\frac{l}{r}$ is used the unit stress usually ranges from 12,000 to 14,000.

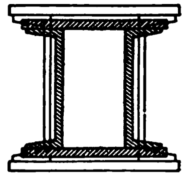


Fig. 277 (*b*).

1. **Illustrative Problem.** — *Channel Column Splice* — Design a splice to connect a "10-inch" channel column, composed of 2–10'' ⊔⊔ 30# and 2 Pls. $12 \times \frac{11}{16}$, to a "12-inch" column, composed of 2–12'' ⊔⊔ 30# and 2 Pls. $14 \times \frac{13}{16}$. The distances from face to face of webs and the gages are found from the table on page 300, as recorded in Fig. 277 (*a*). The distance out to out of cover plates of the lower column, or the distance between splice plates, is $1'1\frac{5}{8}''$, so the distance out to out of reinforcing plates on the upper column should be $1'1\frac{1}{2}''$, in order to leave a clearance of $\frac{1}{16}''$ on each side. Since the distance out to out of cover plates is $11\frac{3}{8}''$, the thickness of the reinforcing plates on each side of the column is $1\frac{1}{16}'' = (1'1\frac{1}{2}'' - 11\frac{3}{8}'') \div 2$, which would be subdivided into $\frac{1}{2}''$ and $\frac{9}{16}''$ to avoid the use of metal thicker than $\frac{3}{4}''$ on account of punching. The distances from back to back of webs is found by subtracting the web thicknesses from the distances out to out of webs found above, and the thickness of the reinforcing plates on each channel is $\frac{15}{16}'' = (\frac{1}{2} \times 8\frac{1}{4} - \frac{1}{2}) - (\frac{1}{2} \times 6\frac{1}{2} - \frac{11}{16}) - \frac{1}{8}$, which would be subdivided into $\frac{1}{2}''$ and $\frac{7}{16}''$. The least radius of gyration of the upper section may be found to be 3.4 (page 211:3), and the unsupported length between floors is assumed to be 12'0'', from which the allowed unit stress at $16{,}000 - 70\frac{l}{r}$ is found to be about 13,000#/sq. in. The developed stress in one $12 \times \frac{11}{16}$ plate is 107,300# = 8.25 × 13,000, and that in one 10'' ⊔ 30# is 114,700# = 8.82 × 13,000. The six $\frac{3}{4}''$ field rivets in single shear at 10,000#/sq. in. are worth 26,500# (page 309) and the number of shop rivets at 12,000#/sq. in. required to take the balance is $16 = \frac{107{,}300 - 26{,}500}{5300}$. The number required in the lower reinforcing plates is 22 = 114,700 ÷ 5300. Only two rivets are used through the splice angle so that it may be made similar to those used where no reinforcing plates are necessary. The completed splice is shown in the figure.

CHAPTER XLI

PINS

SYNOPSIS: A single cylindrical pin may be used instead of rivets to connect different members at a joint. Such a pin is designed as a beam, but the forces do not, as a rule, lie in the same plane.

1. **Description.** — The joints of bridge trusses may be either riveted or pin-connected, as explained on page 121 : 1. Only one pin is used at a joint in a pin-connected truss and greater flexibility is thus obtained than in a truss with fully riveted joints, because the members are free to turn upon the pin as the truss deflects, without causing secondary stresses. Similar small pins are used in conjunction with loop-rods, U-bolts, and clevises. These small pins may be simply rough or turned bolts with ordinary heads and nuts, or they may be "cotter pins" either

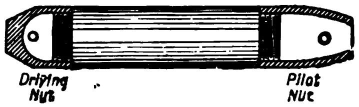


Fig. 278 (*a*).

Fig. 278 (*b*).

with a special head at one end and a hole for a "cotter" at the other (Fig. 279) or with cotters at both ends. The larger bridge pins are turned cylinders with threaded ends of reduced diameter upon which pilot nuts and driving nuts (Fig. 278 (*a*)) are screwed for use in erection. These pins are driven cold without upsetting, the holes in the members being from $\frac{1}{64}''$ to $\frac{1}{32}''$ larger than the diameters of the pins. The pins are held in position by recessed nuts called "Lomas nut" or nuts with washers (Fig. 278 (*b*)) which replace the temporary driving and pilot nuts. Pins over 10″ in diameter may have caps instead of nuts, held in place by small rods passed through holes in the pins.

2. **Pins are designed** as cylindrical beams to resist both bending and shear. Usually the size of the pin will be determined by the bending moment, but the strength of the resulting cross section should be investigated for it may be necessary to increase the size of the pin as a result. Each member which bears upon a pin must furnish sufficient bearing area to properly distribute the stress in that member. This bearing area is the product of the diameter of the pin by the thickness of the metal which bears upon the pin (compare page 230 : 3). The bearing area of riveted members may be increased by the use of reinforcing plates, as explained in the next chapter (page 284), but when this is not feasible the minimum diameter of the pin may be determined by the bearing in some member. Thus the minimum diameter of pin used in any eye bar may be found by dividing the total stress in the bar by the thickness of the bar and by the unit stress in bearing. Often a minimum of 5″ or eight-tenths of the width of the eye bar is specified for bridge work.

3. **Application of Forces.** — For convenience in finding the bending moment on a pin it is usually assumed that the force due to the stress on a member or part of a member acts upon the pin as if concentrated at the center of bearing, i.e., the center of the surface of contact. This assumption is on the side of safety, and the slight increase in the size of the pin is relatively unimportant. The bearing may depend not only upon the thickness of the main part of a member, but also upon the thickness of any reinforcing plates which may be needed (see above). Since

the bearing depends upon the diameter of the pin, the two are interdependent and either the thickness or the diameter must be first assumed and later verified. In this chapter the plate thickness is assumed to be correct. In the case of a small pin for a clevis or a loop-rod, all forces lie in the same plane, and the bending moment is found as in a simple beam. At a joint in a bridge truss, however, the forces are not confined to a single plane, and the position of the point of maximum bending moment is not so apparent.

1. **Design for Bending.** — As in the design of other beams, the maximum bending moment is equated to the resisting moment. For a beam of circular cross section the resisting moment is $0.098fd^3$, where f is the unit stress in bending and d the diameter (page 200:1). The unit stress in bending is usually about 50% greater than for other beams, 24,000#/sq. in. being specified by the American Railway Engineering Association. In order to facilitate the designing of pins, a table of resisting moments for different values of f and d is shown on page 333.

2. **Investigation for Shear.** — In determining whether a pin designed for bending has sufficient area to resist the shear, it should be remembered that the intensity of the shearing stress is not uniform throughout the cross section, but is greatest at the neutral axis. The maximum shear intensity is four-thirds the average intensity, or $\frac{4V}{3\pi r^2} = \frac{16V}{3\pi d^2}$, where V is the maximum shear, r and d the radius and the diameter, respectively, of the pin (page 202 : 1). This maximum shear intensity must not exceed the allowed unit stress in shear, which is usually the same as for shop rivets and usually one-half the unit stress in bending or bearing.

3. **Illustrative Problem.** — *Co-planar Forces.* Design a pin to fasten a 1″ square loop-rod between two ½″ plates, as shown in Fig. 279. The stress in the rod is 16,000#, and the stress in each plate is one-half this amount. The distance from the center of bearing of the rod to the center of bearing of one plate is ¾″ = ½(1 + ½), and the maximum bending moment on the pin at the center is 6000#in. = 8000 × ¾. Using a unit stress in bending of 20,000#/sq. in., the required diameter of the pin may be found from the table on page 333 to be 1½″. The maximum shear intensity is 6000#/sq. in. $= \frac{16 \times 8{,}000}{3\pi(1\frac{1}{2})^2}$, which is safely under the 10,000 allowed.

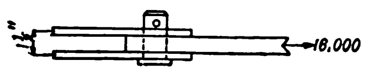


Fig. 279.

4. **Not every pin** in a truss need be designed because it is impractical to use so many different sizes. It is preferable to make several pins alike in order to reduce the number of different eye bars to be made and the number of different pin holes to be bored. It is customary to calculate the size of the pins at the shoe (Fig. 290), at the hip, at the top-chord joint next to the hip, and at the bottom-chord joint nearest the center. Except for very long spans, the remaining pins in the bottom chord may be made like the central pin, and the remaining top-chord pins may be made like that in the joint next to the hip. Often the lower pins can be made like the upper ones.

5. The arrangement of the different members on a pin has its effect upon the size required. The arrangement is usually termed the "**packing.**" Sometimes the results of two or more different arrangements must be compared in order that the best one may be selected. The packing on a pin is made symmetrical, and therefore members which are composed of eye bars should have an even number of bars, and riveted chord members and posts should have two or more webs. A single stirrup or small counter is sometimes placed at the center. The bars of diagonals are usually placed next to the posts in order to reduce the bending moments. No two eye bars of a single member should be placed in contact as it would be impossible to paint between them. Unless a bar of an opposing member is placed between two bars of one member, the latter should be separated on the pin by a collar at least 1″ wide. The pin is thus lengthened 2″, but the diameter may often be reduced considerably. Unless the bars on a pin are within ¼″ of each other, the space between them should be filled with a washer or collar in order to maintain the proper spacing of the bars in accord with the design. In determining the distances from center to center of bearings used in computing the bending moments, an allowance of at least $\frac{1}{16}$″ should be made between adjacent bars to allow for paint, for scale, and for variation in thickness. A clearance of ¼″ should be left between the bars and the built members, due allowance being made for the heads of rivets in reinforcing plates or other component parts. It is often necessary to flatten

or countersink these rivets. The flanges of channels or angles are often notched to clear the bars, provided the webs are properly reinforced. The packing at one pin is dependent upon that at the adjacent pins because the bars should be kept approximately parallel. A bar should not slope more than $\frac{1}{16}''$ per foot. If it is necessary to use a greater slope the bars should be bent to furnish better bearing on the pins.

1. Since the forces which act upon a pin in a truss do not lie in the same plane, each force should be resolved into **horizontal and vertical components.** The position of the point of maximum bending moment is usually not apparent, so it becomes necessary to find the bending moment due to the horizontal components, and that due to the vertical components at each point of concentration. The resultant bending moment at each point will be the square root of the sum of the squares of the horizontal and the vertical bending moments *at that point.* Care must be taken not to combine the horizontal bending moment at one point with the vertical bending moment at another. The maximum bending moment on a pin will often occur at a point where there is no vertical bending moment. The use of the table of squares on page 332, or the diagram on page 312, is recommended in finding the resultants.

2. **The forces** which act upon a pin must be selected with great care. The values on the stress diagram show the maximum stresses for which each complete member is designed, but these maximum stresses do not occur in all members of a truss simultaneously. The maximum chord stresses are found when the truss is fully loaded, but the maximum web-member stresses are found when the truss is only partially loaded. Care should be taken to use either the maximum chord stresses and the *corresponding* stresses in the web members, or conversely, the maximum stresses in the web members and the corresponding chord stresses. As a rule, the first condition will govern the size of the pin at the shoe and at the hip, the second condition will govern the size of the remaining top-chord pins, but either condition may govern the size of the bottom-chord pins. Counters are not stressed when the main diagonals are stressed, but they must be considered in packing the pin because they will cause increased lever arms which will affect the bending moments. The stresses which act upon a pin at any one time must be in equilibrium. To insure this, only the maximum chord stresses *or* the maximum stresses in the diagonals should be taken from the stress diagram, and the other stresses should be computed to correspond. It is well not to use the stress in a post as given on the diagram because the stress on the pin may differ on account of the method of supporting the floor beam. In a bottom-chord joint governed by maximum chord stresses, these chord stresses are taken from the diagram and divided proportionately among the component parts of the members. A sketch is drawn showing all of these horizontal forces and the horizontal components of the forces in the component parts of the diagonals. The proper magnitudes of the latter forces may be determined from the H equation of equilibrium. The vertical components may be determined from these horizontal components, and the corresponding forces in the component parts of the post may be found from the V equation. Similarly, in a top-chord joint the maximum stress in the diagonal is taken from the diagram, and the corresponding stresses in the other members are computed. In case the adjacent top-chord members are in the same straight line, only the difference in their stresses need be found, since the members bear on opposite sides of the pin and cause no bending.

3. **Computation.** — Since the forces on a pin are symmetrically placed, it is necessary to determine the bending moments on only one-half of the pin, care being taken to count only one-half of a force at the center. The bending moment at one point may be best found from the bending moment at the preceding point by adding algebraically the product of the shear for a section between the points by the distance between the points (page 189:1). It is convenient to arrange the computation in tabular form as shown in the problems which follow. When finding the resultant bending moment from the two components it should be remembered that the horizontal bending moment is constant between the horizontal force nearest the center and the corresponding force on the opposite side of the center. This should be obvious because the shear is zero. The actual position of the bars on a pin may vary slightly from the spacing used in the design, and it is therefore consistant to use lever arms to the nearest $\frac{1}{8}''$, and shears and bending moments to the nearest thousand pounds or pound-inches.

4. **Illustrative Problem.** — *Bottom-chord Pin.* Design the pin at the joint $L3$ of the truss shown in Fig. 281 (a). Let us assume that the size of

the pin will be determined when the chord stresses are maximum, and that the arrangement of the bars can be selected without regard to the other joints of the truss. Let us assume also that the reinforcing plates

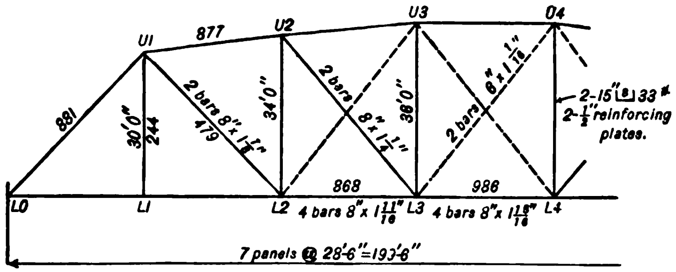


Fig. 281 (a).

on the post have been determined from an assumed size of pin, and that the resulting size is the same as assumed. The results of the two arrangements shown in the figures will be compared. In both arrangements the bars of the main diagonal are placed next to the post, and the counters between the diagonals and the chord bars. Inclined bars may be differentiated from the horizontal bars by section lines as shown. The smaller chord bar is placed at the end, and the other chord bars are alternated in the first arrangement, or placed as shown in the second arrangement with a 1″ collar between the two larger bars. The stress in thousands of pounds in each of the four $8 \times 1\frac{11}{16}$ bars is $217 = 868 \div 4$, and in each of the $8 \times 1\frac{15}{16}$ bars is $247 = 986 \div 4$. In order to satisfy the H equation, the horizontal component in the main diagonal must be $60 = 2 \times 247 - 2 \times 217$. The corresponding vertical component is $71 = 60 \times \frac{34.0}{28.5}$; and the force in one-half the post must be the same. These forces are shown in the small sketches, together with the lever arms which are found as follows:

$$1\tfrac{7}{8}'' = \tfrac{1}{2}(1\tfrac{11}{16} + 1\tfrac{15}{16}) + \tfrac{1}{16}$$
$$2\tfrac{3}{4}'' = \tfrac{1}{2}(1\tfrac{15}{16} + 1\tfrac{1}{4}) + 1\tfrac{1}{16} + 2 \times \tfrac{1}{16}$$
$$1\tfrac{1}{4}'' = \tfrac{1}{2}(1\tfrac{1}{4} + \tfrac{1}{2} + \tfrac{7}{16}) + \tfrac{1}{4} \text{ (the rivets being countersunk)}$$
$$3'' = 1\tfrac{15}{16} + 1 + \tfrac{1}{16}$$

First Arrangement

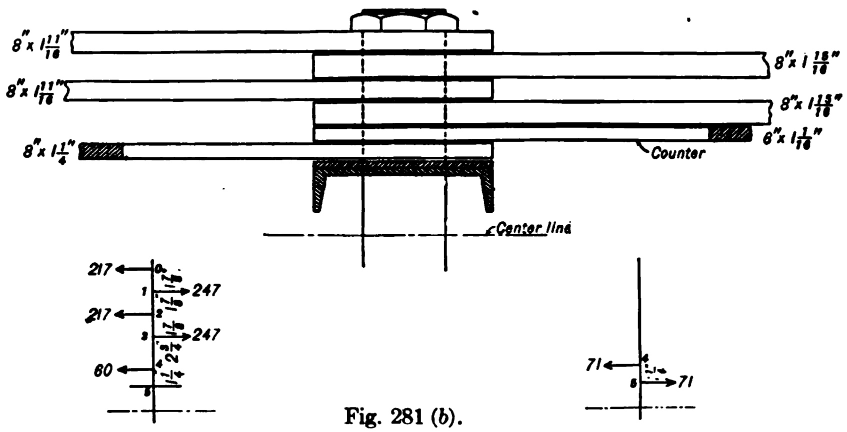


Fig. 281 (b).

Horizontal Components				Point of Moments	Vertical Components			
Shear	Lever Arm	Product	Bending Moment		Shear	Lever Arm	Product	Bending Moment
1000#	In.	1000# in.	1000# in.		1000#	In.	1000# in.	1000# in.
−217	$1\frac{7}{8}$	−407	−407	1				
+30	$1\frac{7}{8}$	+56	−351	2				
−187	$1\frac{7}{8}$	−351	−702	3				
+60	$2\frac{3}{4}$	+165	−537	4				
0	. . .	. . .	−537	5	−71	$1\frac{1}{4}$	−89	−89

The maximum bending moment for this arrangement is 702 at point 3, since this is obviously greater than the resultant $\sqrt{537^2 + 89^2}$ at point 5. It is unnecessary to carry the solution further until the results of the second arrangement are obtained.

Second Arrangement

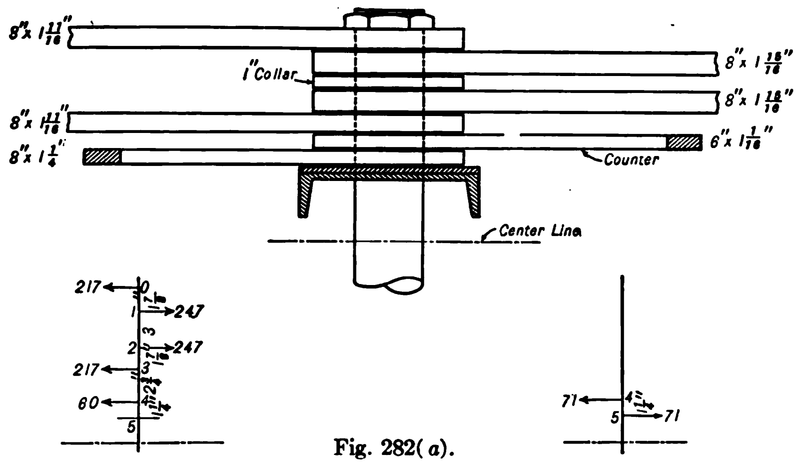


Fig. 282(*a*).

Horizontal Components				Point of Moments	Vertical Components			
Shear	Lever Arm	Product	Bending Moment		Shear	Lever Arm	Product	Bending Moment
1000#	In.	1000# in.	1000# in.		1000#	In.	1000# in.	1000# in.
−217	$1\frac{7}{8}$	−407	−407	1				
+ 30	3	+ 90	−317	2				
+277	$1\frac{7}{8}$	+519	+202	3				
+ 60	$2\frac{3}{4}$	+165	+367	4				
0	...	...	+367	5	−71	$1\frac{1}{4}$	−89	−89

The maximum bending moment for this arrangement is 407 at point 1, since this is more than the resultant $\sqrt{367^2 + 89^2}$ at point 5. This arrangement gives a smaller bending moment and is therefore adopted. Using a unit stress in bending of 24,000#/sq. in., we find from the table on page 333 that a $5\frac{3}{4}''$ pin is required for a bending moment of 407,000# in. The maximum intensity of shear is $14{,}200 = \dfrac{16 \times 277{,}000}{3\,\pi\,(5\frac{3}{4})^2}$. This exceeds the allowed value 12,000, so the diameter must be increased to $6\frac{1}{4}'' = \sqrt{\dfrac{16 \times 277{,}000}{3\,\pi\; 12{,}000}}$. This is less than the diameter required to satisfy the bending moment of the first arrangement and hence is used. In neither arrangement does the presence of the counter increase the size of the pin. If the maximum bending moment was found at point 4 or 5, it might be desirable to place the counters between the channels of the post, cutting the flanges if necessary, in order to reduce the lever arm between points 3 and 4.

1. **Illustrative Problem.** — *Pin at Hip.* Design the pin at the joint $U1$ of the truss shown in Fig. 281 (*a*). Let us assume that the reinforcing plates on the members which meet at this point have been designed from an assumed diameter of pin which proves to be correct, and that the arrangement on the pin is as shown in Fig. 282 (*b*). The outer reinforcing plate of the top chord and the inner plate of the end post are extended to hold the pin in position against shock and to protect the joint from the weather. The centers of bearing of these two mem ers are therefore not quite opposite. The diagonal is placed between the end post and the hip vertical with ample clearance for countersunk rivets in each so the rivets need not be chipped. As a rule only the full load which causes maximum stresses in the end post, the top chord, and the hip vertical need be considered. From the panel lengths and depths the lengths of the members may be calculated, and the components of the maximum stresses in these members may be found by proportion, thus:

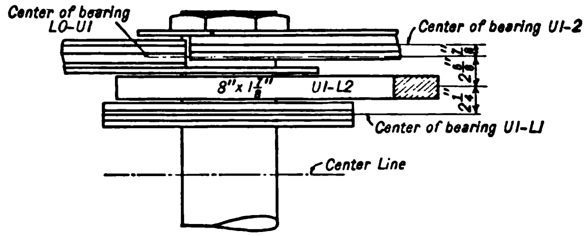


Fig. 282 (*b*).

Horizontal Components

$$EP = 606 = 881 \times \frac{28.5}{41.4}$$

$$U1\text{–}2 = 866 = 877 \times \frac{28.5}{28.8}$$

$$L1\,U1 = 0$$

Vertical Components

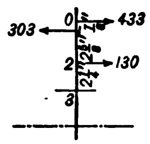
$$EP = 638 = 881 \times \frac{30.0}{41.4}$$

$$U1\text{–}2 = 120 = 877 \times \frac{4.0}{28.8}$$

$$L1\,U1 = 244$$

Since each of these members bears upon the pin at two points of concentration the above values should be divided by 2, and recorded on the sketch as shown. The corresponding components in each bar of the diagonal *L*2 *U*1 may be found from the H and V equations of equilibrium.

From the bending moments tabulated on this page the maximum bending moment is found to be $1154 = \sqrt{720^2 + 902^2}$ at point 3. At 24,000# /sq. in. this requires an 8″ pin. The maximum shear is $437 = \sqrt{433^2 + 60^2}$, and the maximum shear intensity is $11{,}600\# = \dfrac{16 \times 437{,}000}{3\,\pi\,8^2}$ which is less than the allowed 12,000.



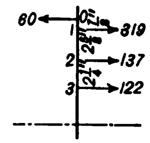


Fig. 283.

Horizontal Components				Point of Moments	Vertical Components			
Shear	Lever Arm	Product	Bending Moment		Shear	Lever Arm	Product	Bending Moment
1000#	In.	1000# in.	1000# in.		1000#	In.	1000# in.	1000# in.
−433	7/8	−379	−379	1	+60	7/8	+53	+53
−130	2 5/8	−341	−720	2	−259	2 5/8	−680	−627
0	. . .	. . .	−720	3	−122	2 1/4	−275	−902

CHAPTER XLII

REINFORCING PLATES

SYNOPSIS: The webs of riveted members of pin-connected trusses must be reinforced at the ends in order that they may properly transmit the stresses to the pins. The method of designing reinforcing plates is here shown.

1. **When Used.** — Reinforcing plates are used to strengthen the weaker parts of members in order to fully develop the strength of the remaining parts. Usually such reinforcement occurs at the ends of the members. Reinforcing plates are used on girder webs (page 266: 3), at the splices of certain office-building columns (page 276: 4), at the joints of pin-connected trusses, and at similar places. The method of designing the reinforcing plates of pin-connected riveted members is illustrated in this chapter. These plates are often termed "pin plates" and their design typifies the design of all reinforcing plates.

2. **Type of Member.** — A compression member in the chord of a simple pin-connected truss is usually composed of two channels with one cover plate (Fig. 122), or of two or more web plates with angles and one or more top cover plates (Fig. 128). A compression web member is often composed of two channels. Each end of these members bears against a cylindrical pin which is located at or near the center lines of the webs. Since no other part of the member bears against the pin the whole stress must reach the pin through the webs, which must be reinforced to furnish the necessary bearing. Most of the tension members of a pin-connected truss are composed of eye bars and these need no reinforcement. A riveted tension member is often used in the two end panels of the bottom chord to provide for a reversal of stress.

3. **Method of Design.** — All reinforcing plates must be designed to furnish the necessary bearing area for the pin. This part of the design consists in finding the thickness of the plates and the number of rivets required to fasten them to the webs. The width of the plates are usually made as large as the available space will accommodate, and the length depends upon the number and the spacing of the rivets. The size of the pin must either be predetermined or assumed (see page 278: 2). The reinforcing plates for a pin-connected tension member composed of channels or plates and angles must not only provide the proper bearing, but also the necessary net section. To prevent failure at the pin, the net section through the pin hole must exceed the net area for which the main member is designed. The amount of this excess is usually specified as 25%. The location of the rivets in the reinforcing plates of a tension member must receive special consideration. Provision must be made to properly reinforce a member which is weakened by having part of the flanges cut away for clearance.

4. **Design for Bearing.** — One-half the total stress in a member with two webs is imparted to the pin through each web and its reinforcing plates. When there are more than two webs, the proportion will be approximately equal to the relative cross-sectional areas included between lines drawn midway between the webs. The bearing area of each web and its reinforcing plates is found by dividing the corresponding stress (one-half or other portion of the total stress) by the unit stress allowed for bearing. The combined thickness of the web and its reinforcing plates is found by dividing this area by the diameter of the pin (compare bearing on rivets, page 230: 3). The web thickness is deducted from this combined thickness, leaving the required thickness of the rein-

forcing plates, which should be increased to the nearest commercial size (a multiple of $\frac{1}{16}''$). The total thickness may be subdivided into as many parts as are best suited to particular conditions. It is usually desirable to place part of the plates on each side of the web to make the rivets more effective. No single plate should exceed $\frac{3}{4}''$ on account of punching.

1. **The number of rivets** in the reinforcing plates should fully develop the plates. If the plates are all on one side of the web, the rivets act in single shear, but if part of the plates are on each side, the rivets act in double shear. The limiting value of each rivet must be selected with care because often the bearing value in a channel web is less than the single-shear value, or the bearing value in a heavy web is more than the double-shear value. For the value of countersunk or flattened rivets, see page 231:2. The bearing value in webs which are not multiples of $\frac{1}{16}''$ may be found by multiplying the decimal thickness by the rivet diameter and by the unit stress in bearing. This should be necessary only when the number of rivets is large; it is usually close enough to roughly interpolate a value from the tables, or to use a value given for the nearest $\frac{1}{16}''$, preference being given to the lower of two values. The developed stress in the plates is the product of the total thickness of the reinforcing plates by the diameter of the pin and by the unit stress in bearing. This value should be used to find the total number of rivets when the plates are all on one side of the web or when the plates are about equally divided on opposite sides. The rivets should distribute the stress proportionately among the component parts of the member as far as practicable. Thus enough rivets should connect the reinforcing plates to the bottom angle to develop the stress in the angle, and rivets in the top angle should develop the angle and part of the cover plate.* See Fig. 128. When more than one plate is used on the same side of the web, or when the plates on opposite sides differ considerably in thickness, the plates may be made of different lengths because it is unnecessary that they contain the same number of rivets. The stress in each plate is proportional to its thickness and equal to the product of the thickness by the diameter of the pin and by the unit stress in bearing. The plates on opposite sides of the web may be considered separately. There must be enough rivets through the outer plate to develop that plate, there must be sufficient rivets through the two outer plates to develop both plates, and so on, the number in the plate next to the web being determined by the stress in all the plates on that side of the web. The rivets do not all have the same limiting value, however. Those which pass through plates on only one side of the web may be counted in single shear, unless the bearing value in a channel web is less. One-half the value of the rivets which pass through plates on both sides of the web may be counted in developing the plates on each side. This will be one-half * the bearing value unless the thickness of the web, or the web and the angles through which part of the rivets may pass, is great enough to develop double shear, in which case the half value equals the single-shear value. The thicker plate should be placed next to the web. Each plate is cut at right angles to the axis of the member. The length of each plate should be as great as the width, or at least three-quarters of the width, in order to fully develop the rivets near the edges. In bridges it is customary to let a thin outer plate extend around the pin, as in Fig. 127, in order to cover the joint between members and to hold the members in position during erection. Rivets in tension members, and in compression members where the pin is not at the extreme end, should meet the added requirements of page 286:2.

2. **Illustrative Problem.** — *Compression Member.* Design the reinforcing plates at the end of a compression member composed of two 12″ ⊔⊔ 30# and one cover plate $14 \times \frac{3}{8}$. The maximum stress in the whole member is 342,000#. Use a 4″ pin and $\frac{3}{4}''$ rivets, with a unit stress in bearing of 24,000#/sq. in.

$$171{,}000\# = 342{,}000 \div 2 = \text{stress in each web and its reinforcement}$$

$$1.78'' = \frac{171{,}000}{4 \times 24{,}000} = \text{combined thickness of web and plates}$$

$$1\tfrac{5}{16}'' = 1.27'' = 1.78 - 0.51 = \text{thickness of reinforcing plates}$$

$$126{,}000\# = 1\tfrac{5}{16} \times 4 \times 24{,}000 = \text{developed stress in plates.}$$

Part of the plates should be placed on each side of the web, preferably, the inner ones extending the full depth of the channel, the outer ones

* For illustrative problems see Johnson-Bryan-Turneaure's "Modern Framed Structures," Part III, John Wiley and Sons, Inc., New York.

* More strictly, the bearing value is divided in proportion to the relative thickness of the plates on the opposite sides of the web, but this is usually not necessary, and it is not consistent with the usual method of designing riveted joints.

being limited to 10″ to clear the flanges. If the plates are about equally divided, as shown in Fig. 286 (*a*), the rivets are limited by the bearing in the $\frac{1}{2}$″ channel web, and the total number required is $14 = 126{,}000 \div 9000$. If a $\frac{9}{16}$″ plate is used on the inside and two $\frac{3}{8}$″ plates on the outside, as shown in Fig. 286 (*b*), the rivet value counted on each side of the web is $4500 = 9000 \div 2$ where plates are on both sides, and 5300 where on only one side.

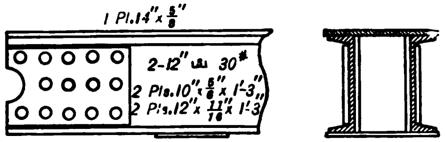


Fig. 286 (*a*).

The number required in the outer $\frac{3}{8}$″ plate is $8 = \dfrac{\frac{3}{8} \times 4 \times 24{,}000}{4500}$. The number required in the $\frac{9}{16}$″ plate is $12 = \dfrac{\frac{9}{16} \times 4 \times 24{,}000}{4500}$. These 12 rivets provide also for an equal stress in the two $\frac{3}{8}$″ plates so that the additional number in single shear is $4 = \dfrac{(\frac{3}{4} - \frac{9}{16})\, 4 \times 24{,}000}{5300}$. Since 12 rivets cannot be arranged to advantage 14 are used in the $\frac{9}{16}$″ plate. By this arrangement it is unnecessary to use $16 = 12 + 4$ rivets in the other $\frac{3}{8}$″ plate because by extending the inner plate all rivets pass through plates on both sides, and 14 in bearing are sufficient for the total stress, as found above for two plates.

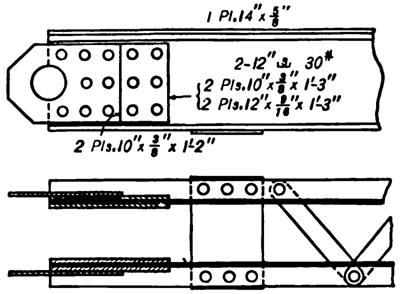


Fig. 286 (*b*).

1. **Design for Tension.** — The reinforcing plates of pin-connected tension members are designed for bearing, as on page 284: 4, but they must also satisfy another requirement. The pin tends to tear out toward the end of the member, and unlike a compression member the whole bearing is on the outer half of the pin. As far as bearing is concerned the reinforcing plates could be placed entirely beyond the pin, where they would act as in a compression member, but the strength of the main member at the pin would not be sufficient to transmit the stress. This part of the member must be reinforced, and logically the same reinforcing plates are extended for this purpose. The plates should be made thick enough for tension as well as for bearing. According to a common clause in the specifications, the total net area of cross section through the pin hole of a tension member must exceed by at least 25% the net area for which the main member is designed. The latter net area is usually found through that row of rivets in the reinforcing plate which is nearest the center of the member (page 208: 2). Furthermore, the member must extend beyond the pin far enough so that the net area of cross section between the pin and the end of the member, parallel to the axis, should not be less than the net area of the main member. In determining the net area at the pin, the diameter of the hole is taken equal to the nominal diameter of the pin, although it is from $\frac{1}{64}$″ to $\frac{1}{32}$″ greater. Due allowance should be made for any reduction in area on account of flanges of channels or angles being cut to clear eye bars or other members.

2. **The rivets in a tension member** should satisfy other requirements in addition to those on page 285: 1. When the thickness of the reinforcing plates determined by the net area exceeds that required for bearing, the rivets should fully develop the strength of the plates in tension. Otherwise the total number of rivets is found as on page 285: 1. The rivets which connect the reinforcing plates to the end of a compression member are naturally all placed on the side of the pin nearer the center of the member. This is often necessarily true because the member cannot extend beyond the pin without interfering with an opposing member. In a tension member the rivets must be divided. Enough rivets must be placed in the plates between the pin and the center of the member to transmit the tensile stress in the plates which is required to develop the necessary net section at the pin, as explained in the preceding paragraph. But no more rivets should be so placed than can be developed by the actual tensile strength of the plates at the pin. Between these minimum and maximum values the number of rivets on the side of the pin toward the center of the member is chosen, and the balance of the total number is placed between the pin and the end of the member. When rivets are placed on both sides of the pin in a compression member the distribution should be determined in a similar manner.

1. **Illustrative Problem.** — *Tension Member.* Design the reinforcing plates at the end of a hanger composed of two 8″ ⊔ $13\frac{3}{4}$#, from which a load of 110,000# is suspended by means of a 3″ pin, as shown in Fig. 287. Use a single 6″ plate on each channel, $\frac{3}{4}$″ rivets, and a bearing value of 20,000#/sq. in.

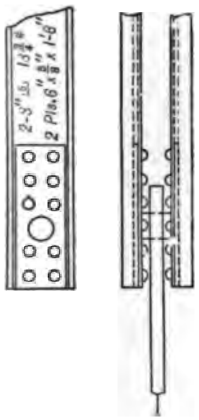


Fig. 287.

55,000# = 110,000 ÷ 2 = stress in each web and its reinforcement

$0.92'' = \dfrac{55,000}{3 \times 20,000}$ = combined thickness of web and plates

$\frac{5}{8}'' = 0.61'' = 0.92 - 0.31$ = thickness of reinforcing plate required for bearing

4.38 sq. in. $= (4.04 - 2 \times \frac{7}{8} \times 0.31)1.25$ = net area required at pin

$\frac{7}{16}'' = 0.42'' = [4.38 - (4.04 - 3 \times 0.31)] \div (6 - 3)$ = thickness of reinforcing plate required for tension

37,500# $= \frac{5}{8} \times 3 \times 20,000$ = developed stress in bearing

21,000# $= (6 - 3) \times \frac{7}{16} \times 16,000$ = developed stress in tension

30,000# $= (6 - 3) \times \frac{5}{8} \times 16,000$ = strength of plate in tension

9 = 37,500 ÷ 4420 = total number of rivets

5 = 21,000 ÷ 4420 = minimum number of rivets above the pin

6+ = 30,000 ÷ 4420 = maximum " " " " " "

The thickness required for bearing exceeds that required for tension and a 6 × $\frac{5}{8}$ plate is used. In order to develop this plate in bearing a total of 9 rivets must be used. In order to develop the required stress in tension at least 5 rivets must be placed above the pin (toward the center), but not more than 6 can be so placed without over stressing the plate in tension. This maximum number falls between 6 and 7, so the *smaller* number is used. At least 5 and not over 6 rivets should be placed above the pin, the remainder being placed below. Since the rivets are placed in two rows, 6 are placed above and 4 below, an extra rivet being used in order to keep the member symmetrical. The length of the member below the bottom of the pin should be at least $3\frac{3}{4}'' = \dfrac{4.04 - 2 \times \frac{7}{8} \times 0.31}{0.62 + 0.31}$ so that the net area will equal that of the main member. If the thickness of the plate required for tension had been greater than that required for bearing, the total number of rivets and the maximum and minimum numbers above the pin would be identical, and all the rivets would be placed above the pin. In this case, a few extra rivets would be used below the pin to hold the plate in place.

CHAPTER XLIII

BEARING PLATES AND COLUMN BASES

Synopsis: Wherever structural steel is supported by masonry, some provision must be made to distribute the load over the proper area. The design of simple bases is discussed in this chapter.

1. **Type.** — Loads from steel beams, trusses, or columns which rest upon masonry must be distributed over a sufficient area so that the allowed bearing value of the masonry will not be exceeded. Simple rectangular plates of steel or cast iron are used under the ends of beams, roof trusses, and some of the lighter girders. Cast-iron pedestals are used under the heavier girders. Plate and angle shoes with expansion rollers support bridge trusses. Cast-iron bases or steel slabs, in conjunction with grillage beams or reinforced concrete piers, are used under office building columns, while other columns are provided with bases built of plates and angles.

2. **Size of Bearing Plate.** — Standard bearing plates are used at the ends of wall-bearing I-beams and channels under usual conditions for the sake of simplicity. The sizes of these plates are given in the tables of I-beams and channels, pages 298 to 302. Special plates should be designed for beams with relatively large reactions, for roof trusses, and for light plate girders or latticed girders. The required area of the plate is equal to the maximum reaction divided by the bearing value of the masonry. The allowed pressure per square inch varies with the different specifications and with the kinds of masonry. Usual values for concrete are from 400 to 600 pounds per square inch, the latter value being specified by the American Railway Engineering Association. For brick, the values are about one-half as large. The shape of the plate which will best meet the requirements depends upon several factors. The best distribution on the masonry is effected when the two dimensions of the plate are approximately equal, but this is not always feasible. The thickness of the plate depends upon the distance the plate projects beyond the edges of the beam, and this should be kept as small as practical. Usually the plate does not extend beyond the end of the beam, and the shorter dimension of the plate determines the amount the beam projects upon the supporting wall. If this is too large, the length of the beam is unnecessarily long, and if too short, dangerous cracks may develop in the wall. The bearing plates of light trusses and girders are usually anchored to the masonry, and the plate must be long enough to provide the necessary edge distances beyond the bolt holes when the anchor bolts are placed far enough from the edges of the angles to permit the turning of the nuts. It may seem wise at times to make the area of the plate somewhat greater than that required, but the thickness may be made correspondingly less because the developed unit pressure on the masonry is reduced.

3. **The thickness of a bearing plate** should be such that the bearing value of the masonry may be developed at every point. The thickness is determined by the maximum projection of the plate beyond the edge of the superimposed metal. This portion of the plate is treated as a cantilever beam with a uniformly distributed pressure on the under side equal to the developed bearing pressure on the masonry. The maximum bending moment on this portion of the plate occurs at the edge of the superimposed metal. An expression for the thickness may be derived, as follows (see Fig. 288):

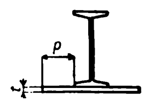


Fig. 288.

Let b = the developed unit stress in bearing on the masonry, i.e., the total load divided by the area at the plate, in pounds per square inch,

f = the allowed unit stress in bending on the extreme fibers of the plate, in pounds per square inch,

p = the projection of the plate beyond the superimposed metal, in inches,

c = the width of the portion of the plate considered, in inches,

t = the required thickness of the plate, in inches.

Since the pressure and the resisting moment are both proportional to c, its value does not affect the result.

bc = the pressure per linear inch, uniformly distributed,

$pbc \times \frac{p}{2}$ = the maximum bending moment in pound-inches (page 187:1),

$\frac{1}{6}fct^2$ = the resisting moment in pound-inches (page 199:3).

By equating the resisting moment to the bending moment and solving, we have

$$t = p\sqrt{\frac{3b}{f}}.$$

The diagram on page 316 gives values for t to correspond to different values of p and b when f = 16,000#/sq. in. For cast-iron plates f is usually about 3000#/sq. in. When the load is applied to the plate by means of angles, as in the case of a roof truss or light girder, the combined thickness of the angle and the plate must be sufficient to prevent bending at the edge of the vertical leg of the angle. This combined thickness t' may be found from the above formula by taking p equal to the distance from the edge of the plate to the vertical leg of the angle, as shown by p', Fig. 289 (a). The fillet of the angle is usually neglected. If the thickness of the angle is fixed, the thickness of the plate should be the greater of two values, one the thickness t required at the edge of the angle, and the other the difference between the combined thickness t' and the thickness of the angle.

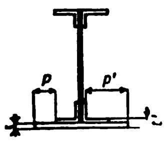


Fig. 289 (a).

1. **Illustrative Problem.** — *Bearing Plate.* Design a bearing plate for a roof truss which rests upon a brick wall, as in Fig. 114 (d). The bottom chord angles are $5 \times 3\frac{1}{2} \times \frac{3}{8}$, and they are separated by a $\frac{3}{8}''$ heel plate. The maximum reaction is 33,000#, and the allowed unit stress on the brick wall is 300#/sq. in. The required area, 110 sq. in. = 33,000 ÷ 300, would be satisfied by a plate $10'' \times 11''$. If $\frac{3}{4}''$ anchor bolts are used, the holes should be punched about 2″ from the edges of the angles to allow for turning the nuts. With an edge distance of about $1\frac{1}{2}''$, the length of the plate must be about $1'2\frac{1}{2}'' = \frac{3}{8} + 2(3\frac{1}{2} + 2 + 1\frac{1}{2})$. Probably it would be better to increase the area accordingly, rather than to reduce the 10″ bearing of the truss. The effect of this increase would be to reduce the developed bearing value b to 228#/sq. in. = 33,000 ÷ (10 × 14.5). The projection beyond the angles is $p = 3\frac{9}{16}'' = \frac{1}{2}(14\frac{1}{2} - \frac{3}{8}) - 3\frac{1}{2}$, and the projection beyond the vertical legs is $p' = 6\frac{11}{16}'' = \frac{1}{2}(14\frac{1}{2} - \frac{3}{8}) - \frac{3}{8}$. From the diagram, the corresponding thickness of the plate alone is $t = \frac{3}{4}''$, and the combined thickness of the plate and the angles is $t' = 1\frac{3}{8}''$. Since the chord angles are only $\frac{3}{8}''$ thick the plate must be at least $1'' = 1\frac{3}{8} - \frac{3}{8}$, and since this exceeds the first value the size of plate adopted is $10 \times 1 \times 1'2\frac{1}{2}''$.

2. **Expansion.** — At one end of a span, two plates may be used instead of one to allow free expansion or contraction under temperature changes. A "masonry plate" rests upon the masonry, and a "sole plate" is riveted to the girder or truss, the surfaces of contact being planed. Slotted holes for the anchor bolts are provided in the upper plate. The combined thickness will be somewhat greater than the thickness of a single plate, because each plate is designed to resist its proportion of the total bending moment. Thus if the two plates are of equal thickness, each would be about 0.7 $= \sqrt{\frac{1}{2}}$ of the thickness required for a single plate.

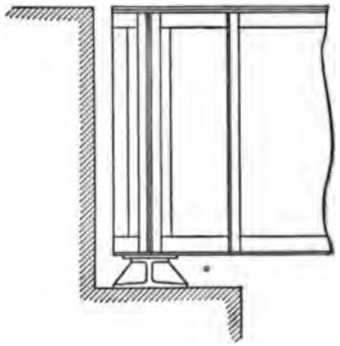


Fig. 289 (b). Cast Pedestal for Bridge Girder.

3. **Pedestals and Shoes.** — Bridge girders are commonly supported by cast-iron or cast-steel pedestals, as shown in Fig. 289 (b). These pedestals are fastened to the masonry by anchor bolts, and the girders are bolted to them. Slotted holes are provided in one end of girders up to about 60 feet in length to allow for expansion, the top of the pedestals and the bottoms of the bearing plates on the girders being

planed. Segmental rollers are placed under the pedestal at one end of girders over 60 feet long. Special hinged shoes or rockers are used at the ends of some of the longer girders to prevent unequal distribution of the load upon the masonry as the girders deflect. Similar hinged shoes are used at the ends of bridge trusses, and roller nests are placed under the shoes at one end, as shown in Fig. 290. For the design of shoes and rollers consult books on Bridge Designing, particularly those listed below.*

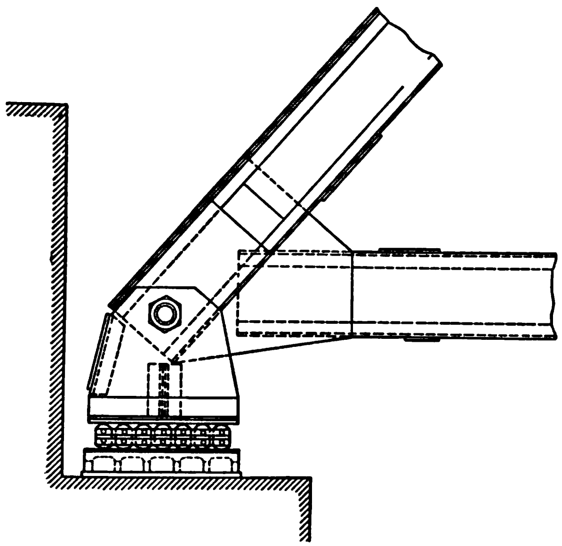


Fig. 290. Typical Bridge-truss Shoe with Rollers.

1. **Column bases** are of two main types which for convenience will be designated according to the principal structures in which they are used, viz.: mill buildings and office buildings. In the former the loads are transmitted largely by rivets, while in the latter by direct bearing. In mill buildings the column loads are comparatively light, and the bases must be anchored to the masonry piers to prevent lateral displacement by accident, and to resist the overturning effect of the wind. Cross bracing can be used only at the ends of a mill building, and the wind pressure on the sides must be resisted largely by the columns acting as beams fixed at the lower ends. The effect of this, combined with the effect of eccentric loading on the column, is to cause unequal distribution of the load on the masonry, and the area of the base must be so proportioned that at no point will the allowed bearing pressure be exceeded. The pressure near one edge of the base may be very small, in fact the upward forces due to wind may exceed the downward forces due to the direct load so that it is necessary to anchor the base to the masonry. In order to make the anchor bolts effective, the base must be securely riveted to the column, and for this reason it is usually built of plates and angles. For light columns, such as shown in Fig. 137, these rivets are designed to transmit the whole column loads. Columns which support crane runways or other moving loads, and columns in which the load exceeds 40,000 pounds are milled so that a portion of the loads are transmitted by direct bearing. A typical crane-girder column base is shown in Fig. 135. In office buildings there are intermediate columns which are rigidly connected at each floor, and the principal wind stresses which reach the basement columns are vertical. The uplift on the windward columns is usually less than the vertical dead loads so that anchors are not required except in tall narrow buildings or towers. From the nature of the building there is slight chance of displacement of the column bases by accident. The bending stresses are small compared to the total loads, and the pressure on the bases is distributed more uniformly than in mill buildings. Cast-iron or cast-steel bases,* such as shown in Fig. 175 (*a*), are commonly used in office-building construction. The bottoms of the columns and the tops of the cast bases are both planed so that practically the entire column loads are transmitted by direct bearing without rivets. Light angles are usually riveted to the end of the column, as in *AB*1, Fig. 133, and these angles are bolted to the cast bases to prevent displacement during erection. The cast bases are grouted on top of reinforced concrete piers or grillage beams, as explained in the next chapter (see Fig. 291), and then they are imbedded in concrete. Column bases are usually standardized by structural companies so that it is unnecessary to design each base. Furthermore, the design depends so much upon the wind stresses, that it seems unwise to attempt further explanation here.†

* Waddell's "Bridge Designing," Vols. I and II, John Wiley and Sons, Inc., New York, and Skinner's "Details of Bridge Construction," Vol. III, McGraw-Hill Book Co., Inc., New York.

* For dimensions of American Bridge Company's standard cast bases see Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Company, Inc., New York.

† For the design of mill-building column bases, see Kirkham's "Structural Engineering," McGraw-Hill Book Co., Inc., New York; for the comparison of different methods of designing anchor bolts, see articles by R. Fleming and E. Godfrey in the *Engineering News*, April 30, 1914, and May 7, 1914; for the design of cast bases for office buildings, see Burt's "Steel Construction," American Technical Society, Chicago.

CHAPTER XLIV

GRILLAGE BEAMS

SYNOPSIS: When it is impractical to extend foundations for heavily loaded columns to bed rock, the footings may be spread over the proper area of soil within a comparatively small depth by means of grillage beams imbedded in concrete.

1. **When Used.** — In providing suitable foundations for office buildings it is often impractical, if not impossible, to extend the footings to bed rock. When the footing of an average office-building column bears directly on the soil a large bearing area is required because of the comparatively small pressure allowed on the soil. If an ordinary masonry pier were designed to satisfactorily distribute the load from the small area at the column base to the large area required on the soil, the pier would be so deep that the cost of the excavation and of the masonry would be prohibitive. The same results may be obtained in much less depth by the use of either steel grillage beams or reinforced concrete slabs. The design of reinforced concrete footings would be out of place in this book. Grillage beams are still in common use since reinforced concrete footings have not yet met with universal favor for various reasons, among them being the uncertain effects of electrolysis and corrosion upon the comparatively small steel areas in the reinforcing rods.

2. **Arrangement.** — Grillage beams are arranged in tiers, as shown in Fig. 291, the beams in one tier being placed at right angles to those in the next tier. The bottom tier rests upon a concrete mat about 12 inches thick. Concrete is placed between the beams of each tier, and ultimately the whole footing, including the cast-iron base, is imbedded in concrete at least 4 inches thick to hold the parts in position and to protect the steel against fire and corrosion. The concrete between the beams acts as inverted arches to complete the bearing area of the tier. The maximum distance in the clear between the beams should be such that the full pressure on the concrete is transmitted to the beams. Unless this clear distance between the edges of the flanges exceeds about one and one-half times the width of each flange it is unnecessary to investigate the strength

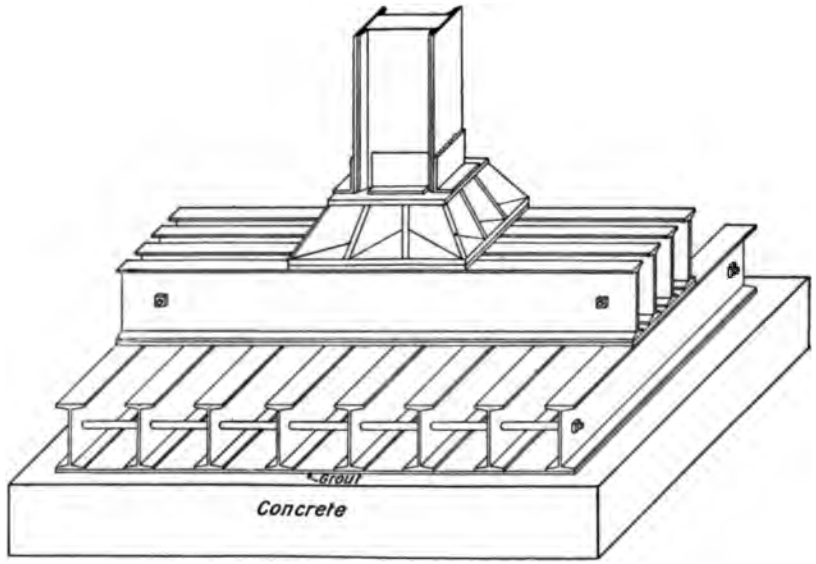


Fig. 291. Grillage Footing (Concrete Filling Not Shown).

of the arch.* The spacing of the beams is usually determined by the number to be placed within a given distance. The distance between

* For the method of investigation, see page 201:3.

flanges should be at least 2½ or 3 inches to permit the placing and the tamping of the concrete. Since it is difficult to insure uniform bearing between the beams of successive tiers when they are placed in contact, a space of about ¾ inch is usually left for grouting. Similarly, if the unit load is not too great, grout is used between the upper beams and the cast base. This facilitates placing the base truly horizontal and at the proper elevation. The allowed pressure on the grout is about 35 tons per square foot, or 500 pounds per square inch. If the column load divided by the area at the bottom of the cast base exceeds this amount, the base must be placed in direct contact with the beams, the bottom of the base being planed. This requires extreme accuracy in setting the beams to furnish uniform bearing for the base at the proper elevation.

1. **Tie Rods.** — Grillage beams are held at uniform distances apart by means of rods and separators. When it is not feasible to use beams with webs thick enough to withstand the tendency to buckle, cast-iron separators or stiffening angles may be used to stiffen the webs. Usually, however, pieces of gas pipe are placed between the webs. Three-fourths inch rods extend through all the beams of each tier, passing through the separators (Fig. 291). Enough rods should be used to resist the thrust of the concrete arches between the beams (preceding paragraph). It is seldom necessary to calculate this thrust. Usually rods are placed about 6 inches from the ends of the beams, and the remaining spaces are subdivided so that the rods are not more than 5 or 6 feet apart. In the smaller beams the rods are placed centrally in the webs; in beams 12 inches or more in depth the rods are used in pairs, the vertical spacing being the same as for cast-iron separators (page 316).

2. **The loads** for which grillage beams are designed are determined by one of several methods.* A grillage foundation will settle as the loads are applied; this cannot be prevented. It is important that the settlement be uniform throughout the structure to prevent cracks in the walls and floors. Under like soil conditions this means that the unit pressure must be the same for all footings. Some engineers design each footing for the total dead and live load, while others design the footing for the critical column and proportion the others from this one according to the ratios that the dead loads bear to the dead load of the critical column, maintaining that the settlement will be proportional to the dead loads because they act at a maximum constantly. The critical column is the one which has the largest ratio of dead load to live load. Other engineers consider one-half (or other fraction) of the live load in conjunction with the dead load in determining the ratios. The live load on the basement column is not the sum of the maximum live loads from each floor, but a certain percentage of this sum depending upon the height of the building.†

3. **The allowed bearing pressure** should be determined by tests made upon the actual material at the site. Records of tests made for nearby buildings are sometimes available. Average values for different characters of soil as specified in different building codes are tabulated in both of the books just referred to. These values are usually expressed in tons per square foot. They may be converted into pounds per square inch by multiplying by $13.89 = 2000 \div 144$, or approximately by multiplying by 14.

4. The extreme dimensions of the **concrete mat** are determined by the area found by dividing the total load by the allowed pressure on the soil. The most desirable shape of mat is a square, but it is not always possible to use a square of sufficient proportions without extending beyond the building lines or interfering with other foundations or pits. Usually a rectangular mat will prove satisfactory. In case it is impracticable to center the grillage under the column, the eccentricity may be overcome by means of a cantilever girder ‡ which extends under an adjacent column. The extreme dimensions of the bottom tier of beams may be made less than the corresponding dimensions of the concrete mat, because the mat itself may be considered to partially distribute the load. The resisting moment of the concrete depends upon the tensile strength in the extreme fiber; this is variable and very small. A projection of 6 inches on each side of a mat 12 inches thick is considered safe, but an increase in this amount would be inadvisable. It may be found that when a

* For comparison, see Jacoby and Davis' "Foundations of Bridges and Buildings," McGraw-Hill Book Co., Inc., New York.

† See copies of the building laws of different cities, or Ketchum's "Structural Engineers' Handbook," McGraw-Hill Book Co., Inc., New York.

‡ See Kidder's "Architects' and Builders' Pocket Book," John Wiley and Sons, Inc., New York.

projection of one-half the depth of the mat is used, the tensile stress in the extreme fiber is three-fourths of the bearing power of the soil.*

1. **Method of Design.** — Grillage beams must be designed to resist bending, buckling, and shearing. The depth of a beam is usually determined by bending, and the web thickness either by bending or buckling. The upper beams are first designed for bending and then their strength is investigated regarding buckling and shear. It may not be necessary to consider either buckling or shear in the design of the lower tiers. Many different arrangements of grillage beams may be designed to satisfy given conditions, because the beams of the different tiers are interdependent. The best arrangement may be selected from the results of several different designs, with due consideration of the comparative costs. Usually two or three tiers will suffice. The extreme width of the top tier of beams is equal to the corresponding dimension of the column base. The length of the beams in one tier is equal to the extreme width of the next tier below. If only two tiers are used, the length of the top beams is equal to the extreme width of the bottom tier. If it is impractical to make the top beams so long, an intermediate tier is used, the length of which is equal to the width of the bottom tier. The width of this intermediate tier, equal to the length of the top tier, is made such that the top beams have equal strength in resisting bending and buckling.

2. **Design for Bending.** — The number of beams to be used in any tier is unknown. For this reason it is convenient to compute the total bending moment on all the beams in the tier, and from this bending moment find the combined section modulus. The proper numbers of beams of different sizes may then be found which will furnish the required section modulus, and the best combination may be selected. The total downward load on each tier of beams is equal to the column load W. It is uniformly distributed throughout the central portion of all the beams for a distance L' equal to the extreme width of the superimposed tier of beams, if any, or to the width of the column base. The beams are supported by upward forces of the same total magnitude (W) but these forces are uniformly distributed throughout the entire length of the beams L. The weight of the beams may be neglected. The maximum bending moment will occur at the center. An expression for the maximum bending moment for all the beams in the tier may be found by the method of page 187:1. The downward forces at the left of the center may be replaced by a single resultant force of $\frac{W}{2}$ acting at a distance of $\frac{L'}{4}$ from the center. Similarly, the resultant of the upward forces is $\frac{W}{2}$ but it acts at a distance of $\frac{L}{4}$ from the center. The bending moment is $\frac{W}{2} \times \frac{L}{4} - \frac{W}{2} \times \frac{L'}{4}$, or

$$\frac{W}{8}(L - L') = M_B\dagger, \text{ or } \frac{W}{8}(l - l') = m_B$$

The combined section modulus of all the beams in the tier may be found by dividing this bending moment by the allowed unit stress in bending. The section modulus should be equaled or exceeded by the product of the number of beams and the section modulus of a single beam.

3. **Investigation for Buckling.** — The portions of the beam webs directly below the superimposed load act as columns and they should be of sufficient thickness to prevent buckling. The usual column formulas (page 211:2) in terms of l and r are inconvenient, but they may be converted into equivalent formulas in terms of d and t, where d is the depth of the beam and t the web thickness. The effective length l may be safely taken as $0.825d$ which is slightly greater than the average tangent distance between the curved fillets connecting the flanges to the web. The formula for unit stress is independent of the area of cross section of the column so we may assume a portion of beam x inches in length. The area of cross section of the column is then tx and the least moment of inertia is $\frac{xt^3}{12}$, whence the least radius of gyration $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{xt^3}{12tx}} = \frac{t}{\sqrt{12}}$. Substituting these values for l and r in the column formula $16{,}000 - 70\frac{l}{r}$ we have $16{,}000 - \frac{70 \times 0.825d\sqrt{12}}{t}$ or

* As in the design of bearing plates (page 288:3) $t = p\sqrt{\frac{3b}{f}}$, whence $f = \frac{3}{4}b$ if $t = 2p$.

† Note that the bending moment is a function of the projection of the beam beyond the superimposed load. The bending moment is equivalent to the bending moment at the center of a simple beam under the same total load, uniformly distributed, for a span equal to this projection (L–L').

$$16{,}000 - 200\frac{d}{t} = \text{the allowed unit stress}$$

per square inch of web section under direct load. Compare page 201:2. This web section is the product of the number of beams in a tier by the thickness of the web and by the distance (in inches) over which the superimposed load is distributed. By some companies this distance is increased by one-quarter or one-half the depth of the beam, part of the web beyond the load being considered effective.

1. **Investigation for Shear.** — The maximum shear on a grillage beam will occur at the outer edge of the superimposed load. Its magnitude may be found by multiplying the distance (in inches) which the beam projects at each end beyond the superimposed load by the upward force per linear inch of beam. This upward force is found by dividing the total column load by the number of beams in the tier and by the length of each beam. The maximum intensity of shear per square inch is found by dividing the above magnitude by the area of cross section of the portion of the web between the flanges (page 202:1). This intensity of shear should not exceed the unit stress in shear allowed by the specifications, as for example 10,000#/sq. in.

2. **Illustrative Problem.** — Design a grillage footing for a column load of 400,000#, using the following unit stresses

$3T$/sq. ft. = 42#/sq. in. = allowed bearing value on soil

500#/sq. in. = " " " " grout

16,000#/sq. in. = unit stress in bending

10,000#/sq. in. = " " " shear

$16{,}000 - 200\frac{d}{t}$#/sq. in. = " " " buckling.

Assuming that a square footing can be used, each side of the 12″ concrete mat should be $98'' = \sqrt{\frac{400{,}000}{42}}$, and allowing a 6″ projection beyond the beams, the length of the bottom beams is $86'' = 98 - 2 \times 6$. Each side of a square cast-iron base should be $28.3'' = \sqrt{\frac{400{,}000}{500}}$ if grout is to be used. Many companies make their bases in multiples of 3″ to minimize the number of different patterns. We will use a 30″ square base.

First Arrangement — Two Tiers

If two tiers of beams are used as in Fig. 291, the length of each is 86″. The load on the upper tier will be distributed over 30″, the width of the column base. The total section modulus is $175 = \frac{400{,}000}{8}(86 - 30) \div 16{,}000$. We can use either 4 – 15″ Is 42#(s = 236) or 4 – 12″ Is 40#(s = 179). The clear distances between flanges are $2.7'' = (30 - 4 \times 5\tfrac{1}{2}) \div 3$, and $3.0'' = (30 - 4 \times 5\tfrac{1}{4}) \div 3$, respectively. Smaller beams need not be tried because more than four would be required and the space between flanges would be too small. The 12″ Is 40# are chosen because they are lighter, and the webs are less liable to buckle. The direct stress which tends to buckle the webs is 7200#/sq. in. $= \frac{400{,}000}{4 \times 30 \times 0.46}$ which is safely under the allowed unit stress of 10,800#/sq. in. $= 16{,}000 - \frac{200 \times 12}{0.46}$. The maximum shear intensity is 7700#/sq. in. $= \frac{400{,}000 \times \frac{1}{2}(86 - 30)}{4 \times 86(12 - 2 \times 1\frac{3}{8})0.46}$ which is less than the 10,000 allowed. The beams selected are therefore satisfactory. The total section modulus for the beams in the lower tier is the same as for those in the upper tier, since both l and l' are the same. The conditions are fulfilled by any one of the following combinations:

Number of Beams	Size	Section Modulus	Total Weight per foot	Clear Distance between Flanges
5	12″ I 31½#	180	158	15.3″ = (86 – 5 × 5.0) ÷ 4
7	10″ I 30#	188	210	8.7″ = (86 – 7 × 4.8) ÷ 6
8	10″ I 25#	195	200	6.9″ = (86 – 8 × 4.7) ÷ 7
10	9″ I 21#	189	210	4.7″ = (86 – 10 × 4.4) ÷ 9
13	8″ I 18#	185	234	2.8″ = (86 – 13 × 4.0) ÷ 12

The 5–12″ Is 31½″ are the lightest and they involve the smallest number of pieces to be handled, but they are too far apart. The 7–10″ Is 30# are also too far apart, and they weigh more than the 8–10″ Is 25#. The latter best meet the requirements and they are strong enough to resist both buckling and shear because $\frac{400{,}000}{8 \times 30 \times 0.31} < 16{,}000 - \frac{200 \times 10}{0.31}$ and

$\frac{400{,}000 \times \frac{1}{2}(86-30)}{8 \times 86 \times 10 \times 0.31} < 10{,}000$. The total weight of the beams for this arrangement is $2580 = (4 \times 40 + 200)86 \div 12$.

Second Arrangement — Three Tiers

It is obvious that the upper-tier beams will be smaller than those in the first arrangement because they are shorter. In order to make the beams of equal strength in resisting bending and buckling, let us first design them to resist buckling and then determine their proper length. The safe loads of different sizes of beams are as follows:

Number of Beams	Size	Safe Load Determined by Resistance to Buckling
4	10″ I 25#	$355{,}000\# = \left(16{,}000 - \frac{200 \times 10}{0.31}\right) 4 \times 30 \times 0.31$
4	9″ I 25#	$570{,}000\# = \left(16{,}000 - \frac{200 \times 9}{0.41}\right) 4 \times 30 \times 0.41$
4	8″ I 20½#	$499{,}000\# = \left(16{,}000 - \frac{200 \times 8}{0.36}\right) 4 \times 30 \times 0.36$

The safe load of the 10″ Is 25# is less than 400,000# so they cannot be used. 4–10″ Is 30# would be sufficient but these would weigh more than the 4–9″ Is 25# so they need not be investigated. 5–8″ Is 18# would be sufficient but they would weigh more than 4–8″ Is 20½#. The latter are the lightest beams which will satisfy the conditions. The distance between flanges is $4.5'' = (30 - 4 \times 4.1) \div 3$. The safe length of these beams in bending may be found by equating an expression for m_B to m_R thus: $\frac{400{,}000}{8}(l - 30) = 4 \times 15.2 \times 16{,}000$, whence $l = 49.5''$ or say $49''$. These beams are also strong enough to resist the shear, because $\frac{400{,}000 \times \frac{1}{2}(49-30)}{4 \times 49(8 - 2 \times \frac{7}{8})\,0.36} < 10{,}000$, hence 4–8″ Is 20½# are selected. The total section modulus for the middle tier is the same as in the bottom tier of the first arrangement, because the l and the l' are the same. The beams may be selected from those shown under the first arrangement, but the same ones cannot be used because they would be too close together. Either 5–12″ Is 31½# or 7–10″ Is 30# could be used, the former being lighter. The distance between flanges is $6'' = (49 - 5 \times 5.0) \div 4$. These 12″ beams are strong enough to resist both buckling and shear because $\frac{400{,}000}{5 \times 30 \times 0.35} < 16{,}000 - \frac{200 \times 12}{0.35}$ and $\frac{400{,}000 \times \frac{1}{2}(86-30)}{5 \times 86(12 - 2 \times 1\frac{3}{8})\,0.35} < 10{,}000$. The total section modulus for the beams in the bottom tier is $116 = \frac{400{,}000}{8}(86 - 49) \div 16{,}000$. The following beams give the required section modulus:

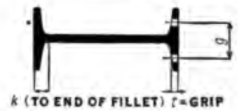
Number of Beams	Size	Section Modulus	Total Weight per foot	Clear Distance between Flanges
7	9″ I 21#	132	147	$9.3'' = (86 - 7 \times 4.4) \div 6$
9	8″ I 18#	128	162	$6.2'' = (86 - 9 \times 4.0) \div 8$
11	7″ I 15#	114	165	$3.9'' = (86 - 11 \times 3.7) \div 10$

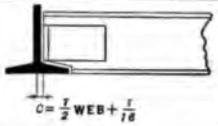
Either the 8″ or the 7″ beams are satisfactory, but the 9″ beams would be too far apart. The 9–8″ Is 18# are chosen because they are lighter. It is usually unnecessary to investigate the strength of the bottom beams for buckling or for shear, because of the relatively large number. The total weight of the beams for this arrangement is $2630\# = (4 \times 20\frac{1}{2} \times 49 + 5 \times 31\frac{1}{2} \times 86 + 9 \times 18 \times 86) \div 12$. This is only slightly more than the weight of the beams for the first arrangement but there are four more beams to handle, and the depth of the excavation and the amount of concrete is more. This can be shown roughly by comparing the extreme depth of steel allowing 1″ between tiers for grouting, thus: $23'' = 12 + 1 + 10$ for the first arrangement and $30'' = 8 + 1 + 12 + 1 + 8$ for the second. In general two tiers are preferred to one, and the first arrangement would be used, as shown in Fig. 291.

TABLES AND DIAGRAMS

For Description, see pages 334–338.

WEIGHTS AND DIMENSIONS OF CARNEGIE I-BEAMS AND AMERICAN BRIDGE COMPANY CONNECTION ANGLES.

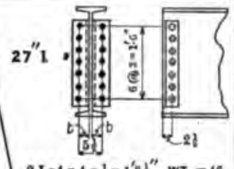


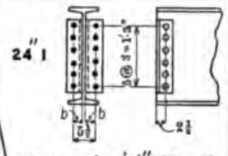


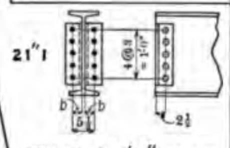
IF THICKNESS OF WEB OR WIDTH OF FLANGE IS MORE THAN $\frac{1}{64}$ ABOVE AN EVEN SIXTEENTH, THE NEXT HIGHER SIXTEENTH IS GIVEN IN THE TABLE BELOW.
r = THE MAXIMUM SIZE OF RIVET OR BOLT USED IN THE FLANGE
p = THE SIZE OF BEARING PLATE, THE WIDTH OF PLATE IS THE LENGTH OF BEARING.
USE $\frac{3}{4}''$ RIVETS IN THE CONNECTION ANGLES
THE WEIGHTS OF CONNECTION ANGLES INCLUDE THE HEADS OF SHOP RIVETS.

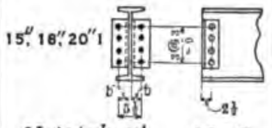
SIZE	WEIGHT	FLANGE	WEB		g	k	r	t	c	b	p
27	90 *	9	.52	$\frac{9}{16}$	5	$2\frac{1}{4}$	1	$\frac{11}{16}$	$\frac{5}{16}$	$2\frac{1}{2}$	16 x 1 x 1'-4"
24	115	8	.75	$\frac{3}{4}$	$4\frac{1}{2}$	$1\frac{7}{8}$	1	$1\frac{1}{16}$	$\frac{7}{16}$	$2\frac{3}{8}$	
	110	$7\frac{15}{16}$	.69	$\frac{11}{16}$					$\frac{7}{16}$	$2\frac{3}{8}$	
	105	$7\frac{7}{8}$	.63	$\frac{5}{8}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	100	$7\frac{1}{4}$	.75	$\frac{3}{4}$	4	$1\frac{5}{8}$	1	$\frac{7}{8}$	$\frac{7}{16}$	$2\frac{3}{8}$	
	95	$7\frac{3}{16}$	.69	$\frac{11}{16}$					$\frac{7}{16}$	$2\frac{3}{8}$	
	90	$7\frac{1}{8}$	.63	$\frac{5}{8}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	85	$7\frac{1}{16}$	.57	$\frac{9}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	80	7	.50	$\frac{1}{2}$					$\frac{5}{16}$	$2\frac{1}{2}$	
	74 *	9	.48	$\frac{1}{2}$	5	2	1	$\frac{9}{16}$	$\frac{5}{16}$	$2\frac{1}{2}$	
21	$60\frac{1}{2}$*	$8\frac{1}{4}$	.43	$\frac{7}{16}$	$4\frac{1}{2}$	$1\frac{3}{4}$	1	$\frac{9}{16}$	$\frac{1}{4}$	$2\frac{9}{16}$	
20	100	$7\frac{5}{16}$	.88	$\frac{7}{8}$	4	$1\frac{3}{4}$	1	$\frac{15}{16}$	$\frac{1}{2}$	$2\frac{5}{16}$	
	95	$7\frac{1}{4}$	.81	$\frac{13}{16}$					$\frac{1}{2}$	$2\frac{5}{16}$	
	90	$7\frac{1}{8}$	.74	$\frac{3}{4}$					$\frac{7}{16}$	$2\frac{3}{8}$	
	85	$7\frac{1}{16}$	.66	$\frac{11}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	80	7	.60	$\frac{5}{8}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	75	$6\frac{7}{16}$	.65	$\frac{11}{16}$	$3\frac{1}{2}$	$1\frac{1}{2}$	1	$\frac{13}{16}$	$\frac{3}{8}$	$2\frac{7}{16}$	
	70	$6\frac{5}{16}$	.58	$\frac{9}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	65	$6\frac{1}{4}$	.50	$\frac{1}{2}$					$\frac{5}{16}$	$2\frac{1}{2}$	
18	90	$7\frac{1}{4}$	.81	$\frac{13}{16}$	4	$1\frac{3}{4}$	$\frac{7}{8}$	$\frac{15}{16}$	$\frac{1}{2}$	$2\frac{5}{16}$	
	85	$7\frac{3}{16}$	.73	$\frac{3}{4}$					$\frac{7}{16}$	$2\frac{3}{8}$	
	80	$7\frac{1}{8}$	.64	$\frac{11}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	75	7	.56	$\frac{9}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	70	$6\frac{1}{4}$	.72	$\frac{3}{4}$	$3\frac{1}{4}$	$1\frac{5}{8}$	$\frac{7}{8}$	$\frac{11}{16}$	$\frac{7}{16}$	$2\frac{3}{8}$	
	65	$6\frac{3}{16}$	.64	$\frac{5}{8}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	60	$6\frac{1}{8}$	.56	$\frac{9}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	55	6	.46	$\frac{1}{2}$					$\frac{5}{16}$	$2\frac{1}{2}$	
	48 *	$7\frac{1}{2}$	.38	$\frac{3}{8}$	4	$1\frac{1}{2}$	1	$\frac{1}{2}$	$\frac{1}{4}$	$2\frac{9}{16}$	
15	75	$6\frac{5}{16}$	.88	$\frac{7}{8}$	$3\frac{1}{4}$	$1\frac{5}{8}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{1}{2}$	$2\frac{5}{16}$	
	70	$6\frac{3}{16}$	.78	$\frac{13}{16}$					$\frac{7}{16}$	$2\frac{3}{8}$	
	65	$6\frac{1}{8}$	.69	$\frac{11}{16}$					$\frac{7}{16}$	$2\frac{3}{8}$	
	60	6	.59	$\frac{5}{8}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	55	$5\frac{3}{4}$	.66	$\frac{11}{16}$	3	$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{8}$	$2\frac{7}{16}$	12 x 1 x 1'-4"
	50	$5\frac{11}{16}$	.56	$\frac{9}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	45	$5\frac{9}{16}$	.46	$\frac{1}{2}$					$\frac{5}{16}$	$2\frac{1}{2}$	
	42	$5\frac{1}{2}$	.41	$\frac{7}{16}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	$37\frac{1}{2}$*	$6\frac{3}{4}$	.33	$\frac{3}{8}$	$3\frac{1}{2}$	$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{7}{16}$	$\frac{1}{4}$	$2\frac{9}{16}$	

CONNECTION ANGLES



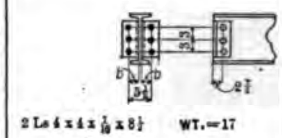


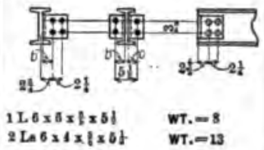


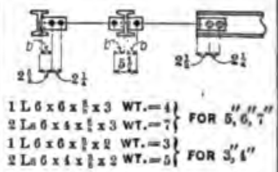


SIZE	WEIGHT	FLANGE	WEB		g	k	r	t	c	b	p
12	55	$5\frac{5}{8}$	.82	$\frac{13}{16}$	3	$1\frac{3}{8}$	$\frac{3}{4}$	$\frac{11}{16}$	$\frac{1}{2}$	$2\frac{5}{16}$	12 x $\frac{3}{4}$ x 1'-0"
	50	$5\frac{1}{2}$	.70	$\frac{11}{16}$					$\frac{7}{16}$	$2\frac{3}{8}$	
	45	$5\frac{3}{8}$	.58	$\frac{9}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	40	$5\frac{1}{4}$	.46	$\frac{1}{2}$					$\frac{5}{16}$	$2\frac{1}{2}$	
	35	$5\frac{1}{8}$	.44	$\frac{7}{16}$	$2\frac{3}{4}$	$1\frac{1}{8}$	$\frac{3}{4}$	$\frac{9}{16}$	$\frac{5}{16}$	$2\frac{1}{2}$	
	$31\frac{1}{2}$	5	.35	$\frac{3}{8}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	28 *	6	.28	$\frac{5}{16}$	$3\frac{1}{4}$	$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{7}{16}$	$\frac{3}{16}$	$2\frac{5}{8}$	
10	40	$5\frac{1}{8}$	.75	$\frac{3}{4}$	$2\frac{3}{4}$	1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{7}{16}$	$2\frac{3}{8}$	8 x $\frac{3}{4}$ x 1'-0"
	35	$4\frac{15}{16}$	.60	$\frac{5}{8}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	30	$4\frac{13}{16}$	.46	$\frac{1}{2}$					$\frac{5}{16}$	$2\frac{1}{2}$	
	25	$4\frac{11}{16}$	.31	$\frac{5}{16}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	$22\frac{1}{4}$*	$5\frac{1}{2}$	.25	$\frac{1}{4}$	3	$1\frac{1}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{16}$	$2\frac{5}{8}$	
9	35	$4\frac{13}{16}$	.73	$\frac{3}{4}$	$2\frac{1}{2}$	1	$\frac{3}{4}$	$\frac{7}{16}$	$\frac{7}{16}$	$2\frac{3}{8}$	8 x $\frac{5}{8}$ x 1'-0"
	30	$4\frac{5}{8}$	.57	$\frac{9}{16}$					$\frac{3}{8}$	$2\frac{7}{16}$	
	25	$4\frac{7}{16}$	.41	$\frac{7}{16}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	21	$4\frac{3}{8}$	.29	$\frac{5}{16}$					$\frac{3}{16}$	$2\frac{5}{8}$	
8	$25\frac{1}{2}$	$4\frac{5}{16}$	.54	$\frac{9}{16}$	$2\frac{1}{4}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{7}{16}$	$\frac{5}{16}$	$2\frac{1}{2}$	8 x $\frac{5}{8}$ x 0'-8"
	23	$4\frac{3}{16}$	.45	$\frac{7}{16}$					$\frac{5}{16}$	$2\frac{1}{2}$	
	$20\frac{1}{2}$	$4\frac{1}{8}$	.36	$\frac{3}{8}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	18	4	.27	$\frac{5}{16}$					$\frac{3}{16}$	$2\frac{5}{8}$	
	$17\frac{1}{2}$*	5	.22	$\frac{1}{4}$	$2\frac{3}{4}$	1	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{16}$	$2\frac{5}{8}$	
7	20	$3\frac{7}{8}$	.46	$\frac{1}{2}$	$2\frac{1}{4}$	$\frac{7}{8}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{5}{16}$	$2\frac{1}{2}$	
	$17\frac{1}{2}$	$3\frac{3}{4}$	.35	$\frac{3}{8}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	15	$3\frac{11}{16}$	.25	$\frac{1}{4}$					$\frac{3}{16}$	$2\frac{5}{8}$	
6	$17\frac{1}{4}$	$3\frac{9}{16}$	.48	$\frac{1}{2}$	2	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{5}{16}$	$2\frac{1}{2}$	6 x $\frac{1}{2}$ x 0'-6"
	$14\frac{3}{4}$	$3\frac{7}{16}$	.35	$\frac{3}{8}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	$12\frac{1}{4}$	$3\frac{3}{8}$	.23	$\frac{1}{4}$					$\frac{3}{16}$	$2\frac{5}{8}$	
5	$14\frac{3}{4}$	$3\frac{5}{16}$	.50	$\frac{1}{2}$	$1\frac{3}{4}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{5}{16}$	$2\frac{1}{2}$	
	$12\frac{1}{4}$	$3\frac{3}{16}$	.36	$\frac{3}{8}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	$9\frac{3}{4}$	3	.21	$\frac{1}{4}$					$\frac{3}{16}$	$2\frac{5}{8}$	
4	$10\frac{1}{2}$	$2\frac{7}{8}$	.41	$\frac{7}{16}$	$1\frac{1}{2}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{1}{4}$	$2\frac{9}{16}$	4 x $\frac{3}{8}$ x 0'-4"
	$9\frac{1}{2}$	$2\frac{13}{16}$	.34	$\frac{3}{8}$					$\frac{1}{4}$	$2\frac{9}{16}$	
	$8\frac{1}{2}$	$2\frac{3}{4}$	.26	$\frac{1}{4}$					$\frac{3}{16}$	$2\frac{5}{8}$	
	$7\frac{1}{2}$	$2\frac{11}{16}$	.19	$\frac{3}{16}$					$\frac{3}{16}$	$2\frac{5}{8}$	
3	$7\frac{1}{2}$	$2\frac{9}{16}$	.36	$\frac{3}{8}$	$1\frac{7}{16}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{4}$	$2\frac{9}{16}$	
	$6\frac{1}{2}$	$2\frac{7}{16}$	.26	$\frac{1}{4}$					$\frac{3}{16}$	$2\frac{5}{8}$	
	$5\frac{1}{2}$	$2\frac{3}{8}$	.17	$\frac{3}{16}$					$\frac{1}{8}$	$2\frac{11}{16}$	

CONNECTION ANGLES

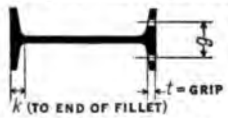


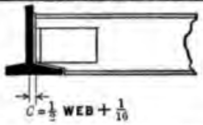




* SUPPLEMENTARY BEAM

WEIGHTS AND DIMENSIONS OF STANDARD I-BEAMS* AND LACKAWANNA CONNECTION ANGLES

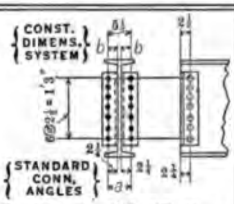




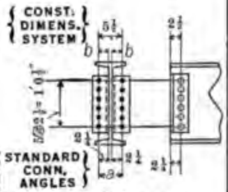
IF THICKNESS OF WEB OR WIDTH OF FLANGE IS MORE THAN $\frac{1}{64}$ ABOVE AN EVEN SIXTEENTH, THE NEXT HIGHER SIXTEENTH IS GIVEN IN THE TABLE BELOW.
r = THE MAXIMUM SIZE OF RIVET OR BOLT USED IN THE FLANGE.
p = THE SIZE OF BEARING PLATE. THE WIDTH OF PLATE IS THE LENGTH OF BEARING.
USE TWO CONNECTION ANGLES WHERE POSSIBLE. $\frac{3}{4}$" RIVETS IN ANGLES.
THE WEIGHTS OF CONNECTION ANGLES INCLUDE THE HEADS OF SHOP RIVETS.

SIZE	WEIGHT	FLANGE	WEB	WEB	g	k	r	t	c	a	$\frac{1}{2}a$	b	p
24	†105	$7\frac{7}{8}$	.63	$\frac{5}{8}$	4	$1\frac{5}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{3}{8}$	$5\frac{1}{8}$		$2\frac{7}{16}$	16 x 1 x 1'4"
	100	$7\frac{1}{4}$	.75	$\frac{3}{4}$					$\frac{7}{16}$	$5\frac{1}{4}$		$2\frac{3}{8}$	
	95	$7\frac{3}{16}$	.69	$\frac{11}{16}$					$\frac{7}{16}$	$5\frac{3}{16}$		$2\frac{3}{8}$	
	90	$7\frac{1}{8}$	.63	$\frac{5}{8}$					$\frac{3}{8}$	$5\frac{1}{8}$		$2\frac{7}{16}$	
	85	$7\frac{1}{16}$	.57	$\frac{9}{16}$					$\frac{3}{8}$	$5\frac{1}{16}$		$2\frac{7}{16}$	
	80	7	.50	$\frac{1}{2}$					$\frac{5}{16}$	5		$2\frac{1}{2}$	
20	100	$7\frac{5}{16}$	.88	$\frac{7}{8}$	4	$1\frac{3}{4}$	$\frac{7}{8}$	$\frac{15}{16}$	$\frac{1}{2}$	$5\frac{3}{8}$		$2\frac{5}{16}$	
	95	$7\frac{1}{4}$	.81	$\frac{13}{16}$					$\frac{1}{2}$	$5\frac{5}{16}$		$2\frac{5}{16}$	
	90	$7\frac{1}{8}$	.74	$\frac{3}{4}$					$\frac{7}{16}$	$5\frac{1}{4}$		$2\frac{3}{8}$	
	85	$7\frac{1}{16}$	.66	$\frac{11}{16}$					$\frac{3}{8}$	$5\frac{3}{16}$		$2\frac{7}{16}$	
	80	7	.60	$\frac{5}{8}$					$\frac{3}{8}$	$5\frac{1}{8}$		$2\frac{7}{16}$	
	75	$6\frac{7}{16}$	.65	$\frac{11}{16}$	$3\frac{1}{2}$	$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{13}{16}$	$\frac{3}{8}$	$5\frac{3}{16}$		$2\frac{7}{16}$	
	70	$6\frac{5}{16}$	.58	$\frac{9}{16}$					$\frac{3}{8}$	$5\frac{1}{16}$		$2\frac{7}{16}$	
	65	$6\frac{1}{4}$	.50	$\frac{1}{2}$					$\frac{5}{16}$	5		$2\frac{1}{2}$	
18	70	$6\frac{1}{4}$	.72	$\frac{3}{4}$	$3\frac{1}{4}$	$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{11}{16}$	$\frac{7}{16}$	$5\frac{1}{4}$		$2\frac{3}{8}$	
	65	$6\frac{3}{16}$	.64	$\frac{5}{8}$					$\frac{3}{8}$	$5\frac{1}{8}$		$2\frac{7}{16}$	
	60	$6\frac{1}{8}$	.56	$\frac{9}{16}$					$\frac{3}{8}$	$5\frac{1}{16}$		$2\frac{7}{16}$	
	55	6	.46	$\frac{1}{2}$					$\frac{5}{16}$	5		$2\frac{1}{2}$	
15	100	$6\frac{13}{16}$	1.18	$1\frac{3}{16}$	$3\frac{3}{4}$	2	$\frac{7}{8}$	$1\frac{1}{16}$	$\frac{11}{16}$	$5\frac{11}{16}$	$2\frac{7}{8}$	$2\frac{1}{8}$	12 x ¾ x 1'4"
	95	$6\frac{11}{16}$	1.09	$1\frac{1}{8}$					$\frac{5}{8}$	$5\frac{5}{8}$	$2\frac{13}{16}$	$2\frac{3}{16}$	
	90	$6\frac{9}{16}$	.99	1					$\frac{9}{16}$	$5\frac{1}{2}$	$2\frac{3}{4}$	$2\frac{1}{4}$	
	85	$6\frac{1}{2}$	.89	$\frac{7}{8}$					$\frac{1}{2}$	$5\frac{3}{8}$	$2\frac{11}{16}$	$2\frac{5}{16}$	
	80	$6\frac{7}{16}$	.81	$\frac{13}{16}$					$\frac{1}{2}$	$5\frac{5}{16}$	$2\frac{11}{16}$	$2\frac{5}{16}$	
	75	$6\frac{5}{16}$	.88	$\frac{7}{8}$	$3\frac{1}{4}$	$1\frac{5}{8}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{1}{2}$	$5\frac{3}{8}$	$2\frac{11}{16}$	$2\frac{5}{16}$	
	70	$6\frac{3}{16}$	78	$\frac{13}{16}$					$\frac{7}{16}$	$5\frac{5}{16}$	$2\frac{5}{8}$	$2\frac{3}{8}$	
	65	$6\frac{1}{8}$	.69	$\frac{11}{16}$					$\frac{7}{16}$	$5\frac{3}{16}$	$2\frac{5}{8}$	$2\frac{3}{8}$	
	60	6	.59	$\frac{5}{8}$					$\frac{3}{8}$	$5\frac{1}{8}$	$2\frac{9}{16}$	$2\frac{7}{16}$	
	55	$5\frac{3}{4}$	.66	$\frac{11}{16}$	3	$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{8}$	$5\frac{3}{16}$	$2\frac{9}{16}$	$2\frac{7}{16}$	
	50	$5\frac{11}{16}$	.56	$\frac{9}{16}$					$\frac{3}{8}$	$5\frac{1}{16}$	$2\frac{9}{16}$	$2\frac{7}{16}$	
	45	$5\frac{9}{16}$	.46	$\frac{1}{2}$					$\frac{5}{16}$	5	$2\frac{1}{2}$	$2\frac{1}{2}$	
	42	$5\frac{1}{2}$	.41	$\frac{7}{16}$					$\frac{1}{4}$	$4\frac{15}{16}$	$2\frac{7}{16}$	$2\frac{9}{16}$	

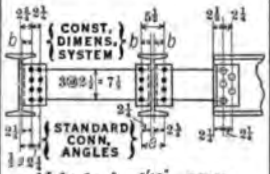
CONNECTION ANGLES



2Ls 4 x 4 x ⅜ x 1'6" WT=32



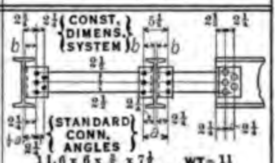
2Ls 4 x 4 x ⅜ x 1'3½" WT=27



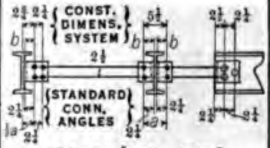
1L 6 x 6 x ⅜ x 0'10" WT=14
2Ls 6 x 4 x ⅜ x 0'10" WT=22

SIZE	WEIGHT	FLANGE	WEB	WEB	g	k	r	t	c	a	$\frac{1}{2}a$	b	p
12	55	$5\frac{5}{8}$	.82	$\frac{13}{16}$	3	$1\frac{3}{8}$	$\frac{3}{4}$	$\frac{11}{16}$	$\frac{1}{2}$	$5\frac{5}{16}$	$2\frac{11}{16}$	$2\frac{5}{16}$	12 x ¾ x 1'0"
	50	$5\frac{1}{2}$	.70	$\frac{11}{16}$					$\frac{7}{16}$	$5\frac{3}{16}$	$2\frac{5}{8}$	$2\frac{3}{8}$	
	45	$5\frac{3}{8}$	.58	$\frac{9}{16}$					$\frac{3}{8}$	$5\frac{1}{16}$	$2\frac{9}{16}$	$2\frac{7}{16}$	
	40	$5\frac{1}{4}$	.46	$\frac{1}{2}$					$\frac{5}{16}$	5	$2\frac{1}{2}$	$2\frac{1}{2}$	
	35	$5\frac{1}{8}$	.44	$\frac{7}{16}$	$2\frac{3}{4}$	$1\frac{1}{8}$	$\frac{3}{4}$	$\frac{9}{16}$	$\frac{5}{16}$	$4\frac{15}{16}$	$2\frac{1}{2}$	$2\frac{1}{2}$	
	$31\frac{1}{2}$	5	.35	$\frac{3}{8}$					$\frac{1}{4}$	$4\frac{7}{8}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
10	40	$5\frac{1}{8}$	.75	$\frac{3}{4}$	$2\frac{5}{8}$	1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{7}{16}$	$5\frac{1}{4}$	$2\frac{5}{8}$	$2\frac{3}{8}$	8 x ⅝ x 1'0"
	35	$4\frac{15}{16}$	.60	$\frac{5}{8}$					$\frac{3}{8}$	$5\frac{1}{8}$	$2\frac{9}{16}$	$2\frac{7}{16}$	
	30	$4\frac{13}{16}$	.46	$\frac{1}{2}$					$\frac{5}{16}$	5	$2\frac{1}{2}$	$2\frac{1}{2}$	
	25	$4\frac{11}{16}$	.31	$\frac{5}{16}$					$\frac{1}{4}$	$4\frac{13}{16}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
9	35	$4\frac{13}{16}$	.73	$\frac{3}{4}$	$2\frac{1}{2}$	1	$\frac{3}{4}$	$\frac{7}{16}$	$\frac{7}{16}$	$5\frac{1}{4}$	$2\frac{5}{8}$	$2\frac{3}{8}$	
	30	$4\frac{5}{8}$	.57	$\frac{9}{16}$					$\frac{3}{8}$	$5\frac{1}{16}$	$2\frac{9}{16}$	$2\frac{7}{16}$	
	25	$4\frac{7}{16}$	.41	$\frac{7}{16}$					$\frac{1}{4}$	$4\frac{15}{16}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
	21	$4\frac{3}{8}$	.29	$\frac{5}{16}$					$\frac{3}{16}$	$4\frac{13}{16}$	$2\frac{3}{8}$	$2\frac{5}{8}$	
8‡	$25\frac{1}{2}$	$4\frac{5}{16}$	.54	$\frac{9}{16}$	$2\frac{1}{4}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{7}{16}$	$\frac{5}{16}$	$5\frac{1}{16}$	$2\frac{1}{2}$	$2\frac{1}{2}$	8 x ⅝ x 8
	23	$4\frac{3}{16}$	.45	$\frac{7}{16}$					$\frac{5}{16}$	$4\frac{15}{16}$	$2\frac{1}{2}$	$2\frac{1}{2}$	
	$20\frac{1}{2}$	$4\frac{1}{8}$	.36	$\frac{3}{8}$					$\frac{1}{4}$	$4\frac{7}{8}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
	18	4	.27	$\frac{5}{16}$					$\frac{3}{16}$	$4\frac{13}{16}$	$2\frac{3}{8}$	$2\frac{5}{8}$	
7	20	$3\frac{7}{8}$	.46	$\frac{1}{2}$	$2\frac{1}{4}$	$\frac{7}{8}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{5}{16}$	5	$2\frac{1}{2}$	$2\frac{1}{2}$	
	$17\frac{1}{2}$	$3\frac{3}{4}$	.35	$\frac{3}{8}$					$\frac{1}{4}$	$4\frac{7}{8}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
	15	$3\frac{11}{16}$	.25	$\frac{1}{4}$					$\frac{3}{16}$	$4\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{5}{8}$	
6	$17\frac{1}{4}$	$3\frac{9}{16}$	.48	$\frac{1}{2}$	2	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{5}{16}$	5	$2\frac{1}{2}$	$2\frac{1}{2}$	6 x ½ x 6
	$14\frac{3}{4}$	$3\frac{7}{16}$	.35	$\frac{3}{8}$					$\frac{1}{4}$	$4\frac{7}{8}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
	$12\frac{1}{4}$	$3\frac{3}{8}$	.23	$\frac{1}{4}$					$\frac{3}{16}$	$4\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{5}{8}$	
5	$14\frac{3}{4}$	$3\frac{5}{16}$	.50	$\frac{1}{2}$	$1\frac{3}{4}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{5}{16}$	5	$2\frac{1}{2}$	$2\frac{1}{2}$	
	$12\frac{1}{4}$	$3\frac{3}{16}$	.36	$\frac{3}{8}$					$\frac{1}{4}$	$4\frac{7}{8}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
	$9\frac{3}{4}$	3	.21	$\frac{1}{4}$					$\frac{3}{16}$	$4\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{5}{8}$	
4	$10\frac{1}{2}$	$2\frac{7}{8}$	.41	$\frac{7}{16}$	$1\frac{1}{2}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{1}{4}$	$4\frac{15}{16}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
	$9\frac{1}{2}$	$2\frac{13}{16}$	.34	$\frac{3}{8}$					$\frac{1}{4}$	$4\frac{7}{8}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
	$8\frac{1}{2}$	$2\frac{3}{4}$	.26	$\frac{1}{4}$					$\frac{3}{16}$	$4\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{5}{8}$	
	$7\frac{1}{2}$	$2\frac{11}{16}$	.19	$\frac{3}{16}$					$\frac{3}{16}$	$4\frac{11}{16}$	$2\frac{3}{8}$	$2\frac{5}{8}$	
3	$7\frac{1}{2}$	$2\frac{9}{16}$	36	$\frac{3}{8}$	$1\frac{7}{16}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{4}$	$4\frac{7}{8}$	$2\frac{7}{16}$	$2\frac{9}{16}$	
	$6\frac{1}{2}$	$2\frac{7}{16}$	.26	$\frac{1}{4}$					$\frac{3}{16}$	$4\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{5}{8}$	
	$5\frac{1}{2}$	$2\frac{3}{8}$	.17	$\frac{3}{16}$					$\frac{1}{8}$	$4\frac{11}{16}$	$2\frac{5}{16}$	$2\frac{11}{16}$	

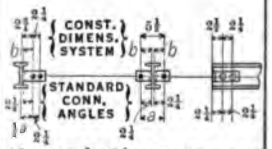
CONNECTION ANGLES



1L 6 x 6 x ⅜ x 7½ WT=11
2Ls 6 x 4 x ⅜ x 7½ WT=17



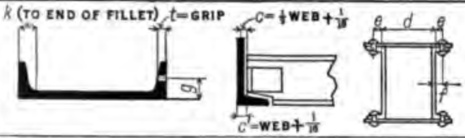
1L 6 x 6 x ⅜ x 5 WT=7
2Ls 6 x 4 x ⅜ x 5 WT=11



1L 6 x 6 x ⅜ x 2½ WT=4 }
2Ls 6 x 4 x ⅜ x 2½ WT=6 } FOR 5" & 6"
1L 6 x 6 x ⅜ x 1½ WT=3 }
2Ls 6 x 4 x ⅜ x 1½ WT=4 } FOR 3" & 4"

*BASED UPON THE STANDARDS OF THE CAMBRIA, LACKAWANNA, PHŒNIX AND JONES AND LAUGHLIN STEEL COMPANIES. †SPECIAL ‡CAMBRIA 8" I's WEIGH $25\frac{1}{4}$, $22\frac{1}{4}$, $20\frac{1}{4}$, AND 18.
CAMBRIA CONNECTION ANGLES AND
PHŒNIX CONNECTIONS ANGLES ARE SIMILAR TO THOSE ON THE OPPOSITE PAGE.

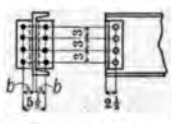
WEIGHTS AND DIMENSIONS OF CARNEGIE CHANNELS AND AMERICAN BRIDGE COMPANY CONNECTION ANGLES



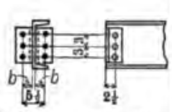
IF THICKNESS OF WEB OR WIDTH OF FLANGE IS MORE THAN $\frac{1}{64}$ ABOVE AN EVEN SIXTEENTH, THE NEXT HIGHER SIXTEENTH IS GIVEN IN THE TABLE BELOW.
r = THE MAXIMUM SIZE OF RIVET OR BOLT USED IN THE FLANGE.
p = THE SIZE OF BEARING PLATE. THE WIDTH OF PLATE IS THE LENGTH OF BEARING.
USE TWO CONNECTION ANGLES WHERE POSSIBLE. $\frac{3}{4}''$ RIVETS IN ANGLES.
THE WEIGHTS OF CONNECTION ANGLES INCLUDE THE HEADS OF SHOP RIVETS.

SIZE	WEIGHT	FLANGE	WEB		g	k	r	t	c	c'	d	e	f	b	p
15	55	$3\frac{13}{16}$	.82	$\frac{13}{16}$	$2\frac{1}{4}$	$1\frac{3}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	9 or 11	$1\frac{3}{4}$	$2\frac{9}{16}$	$2\frac{5}{16}$	12 x 1 x 1'-4"
	50	$3\frac{3}{4}$	.72	$\frac{3}{4}$					$\frac{7}{16}$	$\frac{13}{16}$			$2\frac{1}{2}$	$2\frac{3}{8}$	
	45	$3\frac{5}{8}$	.62	$\frac{5}{8}$					$\frac{3}{8}$	$\frac{11}{16}$			$2\frac{3}{8}$	$2\frac{7}{16}$	
	40	$3\frac{9}{16}$	.52	$\frac{9}{16}$	$1\frac{7}{8}$	$1\frac{3}{8}$	$\frac{3}{4}$	$\frac{11}{16}$	$\frac{5}{16}$	$\frac{5}{8}$			$2\frac{5}{16}$	$2\frac{1}{2}$	
	35	$3\frac{7}{16}$	.43	$\frac{7}{16}$					$\frac{1}{4}$	$\frac{1}{2}$			$2\frac{3}{16}$	$2\frac{9}{16}$	
	33	$3\frac{7}{16}$	.40	$\frac{7}{16}$					$\frac{1}{4}$	$\frac{1}{2}$			$2\frac{3}{16}$	$2\frac{9}{16}$	
13	50	$4\frac{7}{16}$	.79	$\frac{13}{16}$	$2\frac{3}{4}$	$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{9}{16}$	$\frac{7}{16}$	$\frac{7}{8}$	$8\frac{1}{4}$ or $10\frac{1}{4}$	2	$2\frac{13}{16}$	$2\frac{3}{8}$	12 x 1 x 1'-0"
	45	$4\frac{5}{16}$	.68	$\frac{11}{16}$					$\frac{3}{8}$	$\frac{3}{4}$			$2\frac{11}{16}$	$2\frac{7}{16}$	
	40	$4\frac{3}{16}$	.57	$\frac{9}{16}$	$2\frac{1}{2}$	$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{9}{16}$	$\frac{3}{8}$	$\frac{5}{8}$			$2\frac{9}{16}$	$2\frac{7}{16}$	
	37	$4\frac{1}{8}$	.50	$\frac{1}{2}$					$\frac{5}{16}$	$\frac{9}{16}$			$2\frac{1}{2}$	$2\frac{1}{2}$	
	35	$4\frac{1}{16}$	.45	$\frac{7}{16}$					$\frac{5}{16}$	$\frac{1}{2}$			$2\frac{7}{16}$	$2\frac{1}{2}$	
	32	4	.38	$\frac{3}{8}$					$\frac{1}{4}$	$\frac{7}{16}$			$2\frac{3}{8}$	$2\frac{9}{16}$	
12	40	$3\frac{7}{16}$	.76	$\frac{3}{4}$	2	1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{13}{16}$	8 or 10	$1\frac{1}{2}$	$2\frac{1}{4}$	$2\frac{3}{8}$	12 x $\frac{3}{4}$ x 1'-0"
	35	$3\frac{5}{16}$	.64	$\frac{5}{8}$					$\frac{3}{8}$	$\frac{11}{16}$			$2\frac{1}{8}$	$2\frac{7}{16}$	
	30	$3\frac{3}{16}$	.51	$\frac{1}{2}$					$\frac{5}{16}$	$\frac{9}{16}$			2	$2\frac{1}{2}$	
	25	$3\frac{1}{16}$	.39	$\frac{3}{8}$	$1\frac{3}{4}$	1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{7}{16}$			$1\frac{7}{8}$	$2\frac{9}{16}$	
	$20\frac{1}{2}$	$2\frac{15}{16}$	.28	$\frac{5}{16}$					$\frac{3}{16}$	$\frac{3}{8}$			$1\frac{13}{16}$	$2\frac{5}{8}$	
10	35	$3\frac{3}{16}$	.82	$\frac{13}{16}$	2	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	$6\frac{1}{4}$ or $8\frac{1}{4}$	$1\frac{1}{4}$	$2\frac{1}{16}$	$2\frac{5}{16}$	8 x $\frac{3}{4}$ x 1'-0"
	30	$3\frac{1}{16}$	.68	$\frac{11}{16}$					$\frac{3}{8}$	$\frac{3}{4}$			$1\frac{15}{16}$	$2\frac{7}{16}$	
	25	$2\frac{7}{8}$	.53	$\frac{9}{16}$					$\frac{5}{16}$	$\frac{5}{8}$			$1\frac{13}{16}$	$2\frac{1}{2}$	
	20	$2\frac{3}{4}$	.38	$\frac{3}{8}$	$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{7}{16}$	$\frac{1}{4}$	$\frac{7}{16}$			$1\frac{5}{8}$	$2\frac{9}{16}$	
	15	$2\frac{5}{8}$	.24	$\frac{1}{4}$					$\frac{3}{16}$	$\frac{5}{16}$			$1\frac{1}{2}$	$2\frac{5}{8}$	
9	25	$2\frac{13}{16}$	.62	$\frac{5}{8}$	$1\frac{3}{4}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{11}{16}$				$2\frac{7}{16}$	8 x $\frac{5}{8}$ x 1'-0"
	20	$2\frac{11}{16}$	.45	$\frac{7}{16}$					$\frac{5}{16}$	$\frac{1}{2}$				$2\frac{1}{2}$	
	15	$2\frac{1}{2}$	.29	$\frac{5}{16}$	$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{7}{16}$	$\frac{3}{16}$	$\frac{3}{8}$				$2\frac{5}{8}$	
	$13\frac{1}{4}$	$2\frac{7}{16}$	.23	$\frac{1}{4}$					$\frac{3}{16}$	$\frac{5}{16}$				$2\frac{5}{8}$	
8	$21\frac{1}{4}$	$2\frac{5}{8}$	.58	$\frac{5}{8}$	$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{11}{16}$				$2\frac{7}{16}$	8 x $\frac{5}{8}$ x 0'-8"
	$18\frac{3}{4}$	$2\frac{9}{16}$	.49	$\frac{1}{2}$					$\frac{5}{16}$	$\frac{9}{16}$				$2\frac{1}{2}$	
	$16\frac{1}{4}$	$2\frac{7}{16}$	.40	$\frac{7}{16}$					$\frac{1}{4}$	$\frac{1}{2}$				$2\frac{9}{16}$	
	$13\frac{3}{4}$	$2\frac{3}{8}$	.31	$\frac{5}{16}$	$1\frac{1}{4}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{3}{8}$	$\frac{3}{16}$	$\frac{3}{8}$				$2\frac{5}{8}$	
	$11\frac{1}{4}$	$2\frac{1}{4}$	.22	$\frac{1}{4}$					$\frac{3}{16}$	$\frac{5}{16}$				$2\frac{5}{8}$	
7	$19\frac{3}{4}$	$2\frac{1}{2}$	.63	$\frac{5}{8}$	$1\frac{1}{2}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{11}{16}$				$2\frac{7}{16}$	
	$17\frac{1}{4}$	$2\frac{7}{16}$	.53	$\frac{9}{16}$					$\frac{5}{16}$	$\frac{5}{8}$				$2\frac{1}{2}$	
	$14\frac{3}{4}$	$2\frac{5}{16}$	.42	$\frac{7}{16}$	$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{2}$				$2\frac{9}{16}$	
	$12\frac{1}{4}$	$2\frac{3}{16}$	.32	$\frac{5}{16}$					$\frac{1}{4}$	$\frac{3}{8}$				$2\frac{9}{16}$	
	$9\frac{3}{4}$	$2\frac{1}{8}$	.21	$\frac{1}{4}$					$\frac{3}{16}$	$\frac{5}{16}$				$2\frac{5}{8}$	
6	$15\frac{1}{2}$	$2\frac{5}{16}$	.56	$\frac{9}{16}$	$1\frac{3}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{5}{8}$				$2\frac{7}{16}$	6 x $\frac{1}{2}$ x 0'-6"
	13	$2\frac{3}{16}$	.44	$\frac{7}{16}$					$\frac{5}{16}$	$\frac{1}{2}$				$2\frac{1}{2}$	
	$10\frac{1}{2}$	$2\frac{1}{16}$	.32	$\frac{5}{16}$	$1\frac{1}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{3}{8}$				$2\frac{9}{16}$	
	8	$1\frac{15}{16}$	.20	$\frac{3}{16}$					$\frac{3}{16}$	$\frac{1}{4}$				$2\frac{5}{8}$	
5	$11\frac{1}{2}$	$2\frac{1}{16}$	.48	$\frac{1}{2}$	$1\frac{1}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{9}{16}$				$2\frac{1}{2}$	
	9	$1\frac{7}{8}$	.33	$\frac{3}{8}$					$\frac{1}{4}$	$\frac{7}{16}$				$2\frac{9}{16}$	
	$6\frac{1}{2}$	$1\frac{3}{4}$	.19	$\frac{3}{16}$	1	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{3}{16}$	$\frac{1}{4}$				$2\frac{5}{8}$	
4	$7\frac{1}{4}$	$1\frac{3}{4}$	.33	$\frac{5}{16}$	1	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{8}$				$2\frac{9}{16}$	4 x $\frac{3}{8}$ x 0'-4"
	$6\frac{1}{4}$	$1\frac{11}{16}$	.25	$\frac{1}{4}$					$\frac{3}{16}$	$\frac{5}{16}$				$2\frac{5}{8}$	
	$5\frac{1}{4}$	$1\frac{5}{8}$	.18	$\frac{3}{16}$					$\frac{1}{8}$	$\frac{1}{4}$				$2\frac{11}{16}$	
3	6	$1\frac{5}{8}$	.36	$\frac{3}{8}$	$\frac{15}{16}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{7}{16}$				$2\frac{9}{16}$	
	5	$1\frac{1}{2}$	.26	$\frac{1}{4}$					$\frac{3}{16}$	$\frac{5}{16}$				$2\frac{5}{8}$	
	4	$1\frac{7}{16}$	.17	$\frac{3}{16}$					$\frac{1}{8}$	$\frac{1}{4}$				$2\frac{11}{16}$	

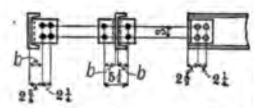
CONNECTION ANGLES



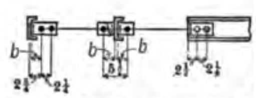
2 Ls 4 x 4 x $\frac{7}{16}$ x $11\frac{1}{2}$ WT.=23



2 Ls 4 x 4 x $\frac{7}{16}$ x $8\frac{1}{2}$ WT.=17

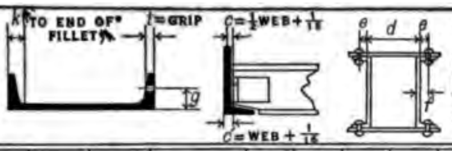


1 L 6 x 6 x $\frac{3}{8}$ x $5\frac{1}{2}$ WT.=8
2 Ls 6 x 4 x $\frac{3}{8}$ x $5\frac{1}{2}$ WT.=13

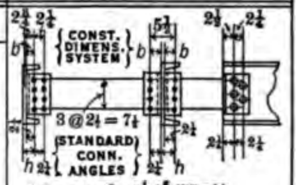
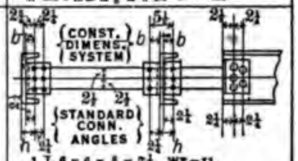
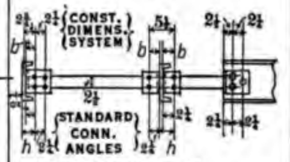
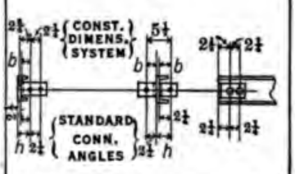


1 L 6 x 6 x $\frac{3}{8}$ x 3 WT.=4 } FOR 5" 6" 7"
2 Ls 6 x 4 x $\frac{3}{8}$ x 3 WT.=7 }
1 L 6 x 6 x $\frac{3}{8}$ x 2 WT.=3 } FOR 3" 4"
2 Ls 6 x 4 x $\frac{3}{8}$ x 2 WT.=5 }

WEIGHTS AND DIMENSIONS OF STANDARD CHANNELS* AND LACKAWANNA CONNECTION ANGLES.



IF THICKNESS OF WEB OR WIDTH OF FLANGE IS MORE THAN 1/64" ABOVE AN EVEN SIXTEENTH, THE NEXT HIGHER SIXTEENTH IS GIVEN IN THE TABLE BELOW.
r = THE MAXIMUM SIZE OF RIVET OR BOLT USED IN THE FLANGE.
p = SIZE OF BEARING PLATE. THE WIDTH OF PLATE IS THE LENGTH OF BEARING.
USE TWO CONNECTION ANGLES WHERE POSSIBLE. ¾" RIVETS IN ANGLES.
THE WEIGHTS OF CONNECTION ANGLES INCLUDE THE HEADS OF SHOP RIVETS.

SIZE	WEIGHT	FLANGE	WEB		g	k	r	t	c	c'	d	e	f	h	b	p	CONNECTION ANGLES
15	55	3 13/16	.82	13/16	2¼	1⅛	¾	⅝	½	⅞	10¼ or 13¼	1½	2 5/16	3 1/16	2⅝	12 x ¾ x 1'4"	 1 L 6 x 6 x ⅝ x 0'10" WT=14 2 Ls 6 x 4 x ⅝ x 0'10" WT=22
	50	3¾	.72	¾					7/16	13/16			2¼	3	2⅝		
	45	3⅝	.62	⅝					⅜	11/16			2⅛	2⅞	2 7/16		
	40	3 9/16	.52	9/16	1⅞	1⅛	¾	11/16	5/16	⅝			2 1/16	2 13/16	2½		
	35	3 7/16	.43	7/16					¼	½			1 15/16	2 11/16	2 9/16		
	33	3⅜	.40	7/16					¼	½			1 15/16	2 11/16	2 9/16		
12	40	3 7/16	.76	¾	2	1	¾	½	7/16	13/16	8¼ or 10¼	1½	2¼	3	2⅝	12 x ⅝ x 1'0"	 1 L 6 x 6 x ⅝ x 7½ WT=11 2 Ls 6 x 4 x ⅝ x 7½ WT=17
	35	3 5/16	.64	⅝					⅜	11/16			2⅛	2⅞	2 7/16		
	30	3 3/16	.51	½					5/16	9/16			2	2¾	2¼		
	25	3 1/16	.39	⅜	1¾	1	¾	½	¼	7/16			1⅞	2⅝	2 9/16		
	20½	2 15/16	.28	5/16					3/16	⅜			1 13/16	2 9/16	2⅝		
10	35	3 3/16	.82	13/16	2	⅞	¾	⅝	½	⅞	6¼ or 9¼	1¼	2 1/16	3 1/16	2 5/16	8 x ⅝ x 1'0"	
	30	3 1/16	.68	11/16					⅜	¾			1 15/16	2 15/16	2 7/16		
	25	2⅞	.53	9/16					5/16	⅝			1 13/16	2 13/16	2½		
	20	2¾	.38	⅜	1½	⅞	¾	7/16	¼	7/16			1⅝	2⅝	2 9/16		
	15	2⅝	.24	¼					3/16	5/16			1½	2½	2⅝		
9	25	2 13/16	.62	⅝	1¾	⅞	¾	⅝	⅜	13/16	6 or 8	1⅜	1¾	2⅞	2 7/16		 1 L 6 x 6 x ⅝ x 5 WT=7 2 Ls 6 x 4 x ⅝ x 5 WT=11
	20	2 11/16	.45	7/16					5/16	½			1 9/16	2 11/16	2½		
	15	2½	.29	5/16	1⅝	⅞	¾	7/16	3/16	⅜			1 7/16	2 9/16	2⅝		
	13¼	2 7/16	.23	¼					3/16	5/16			1⅜	2½	2⅝		
8	21¼	2⅝	.58	⅝	1½	⅞	¾	⅝	⅜	11/16	5¼ or 7¼	1	1⅝	2⅞	2 7/16	8 x ⅝ x 8	
	18¾	2 9/16	.49	½					5/16	9/16			1½	2¾	2½		
	16¼	2 7/16	.40	7/16					¼	½			1 7/16	2 11/16	2 9/16		
	13¾	2⅜	.31	5/16	1¼	⅞	¾	⅜	3/16	⅜			1 5/16	2 9/16	2⅝		
	11¼	2¼	.22	¼					3/16	5/16			1¼	2½	2⅝		
7	19¾	2½	.63	⅝	1½	¾	⅝	⅝	⅜	11/16	4½ or 6¼	1	1⅝	2⅞	2 7/16		
	17¼	2 7/16	.53	9/16					5/16	⅝			1 9/16	2 13/16	2½		
	14¾	2 5/16	.42	7/16	1¼	¾	⅝	⅝	¼	½			1 7/16	2 11/16	2 9/16		
	12¼	2 3/16	.32	5/16					¼	⅜			1 5/16	2 9/16	2 9/16		
	9¾	2⅛	.21	¼					3/16	5/16			1¼	2½	2⅝		
6	15½	2 5/16	.56	9/16	1⅜	¾	⅝	⅝	⅜	⅝	4 or 5	⅞	1 7/16	2 13/16	2 7/16	6 x ½ x 6	 1 L 6 x 6 x ⅝ x 2½ WT=4 } FOR 5" & 6" 2 Ls 6 x 4 x ⅝ x 2½ WT=6 1 L 6 x 6 x ⅝ x 1½ WT=3 } FOR 3" & 4" 2 Ls 6 x 4 x x 1½ WT=4
	13	2 3/16	.44	7/16					5/16	½			1 5/16	2 11/16	2½		
	10½	2 1/16	.32	5/16	1⅛	¾	⅝	⅝	¼	⅜			1 3/16	2 9/16	2 9/16		
	8	1 15/16	.20	3/16					3/16	¼			1 1/16	2 7/16	2⅝		
5	11½	2 1/16	.48	½	1¼	⅝	½	5/16	5/16	9/16				2¾	2½		
	9	1⅞	.33	⅜					¼	7/16				2⅝	2 9/16		
	6½	1¾	.19	3/16	1	⅝	½	5/16	3/16	¼				2 7/16	2⅝		
4	7¼	1¾	.33	5/16	1	⅝	½	5/16	¼	⅜				2 9/16	2 9/16		
	6¼	1 11/16	.25	¼					3/16	5/16				2½	2⅝		
	5¼	1⅝	.18	3/16					⅛	¼				2 7/16	2 11/16		
3	6	1⅝	.36	⅜	15/16	⅝	⅜	¼	¼	7/16				2⅝	2 9/16		
	5	1½	.26	¼					3/16	5/16				2½	2⅝		
	4	1 7/16	.17	3/16					⅛	¼				2 7/16	2 11/16		

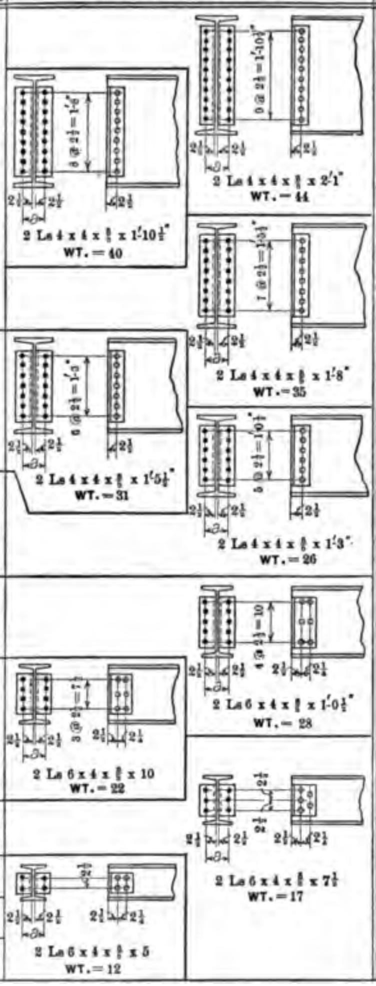
* BASED UPON THE STANDARDS OF THE CAMBRIA, LACKAWANNA, PHOENIX, AND JONES AND LAUGHLIN STEEL COMPANIES.
CAMBRIA CONNECTION ANGLES AND PHOENIX CONNECTION ANGLES ARE SIMILAR TO THOSE ON THE OPPOSITE PAGE.

WEIGHTS AND DIMENSIONS OF BETHLEHEM I-BEAMS AND GIRDER BEAMS

IF THICKNESS OF WEB OR WIDTH OF FLANGE IS MORE THAN 1/64 ABOVE AN EVEN SIXTEENTH, THE NEXT HIGHER SIXTEENTH IS GIVEN IN THE TABLE BELOW.
r = THE MAXIMUM SIZE OF RIVET OR BOLT USED IN THE FLANGE.
p = THE SIZE OF BEARING PLATE. THE WIDTH OF PLATE IS THE LENGTH OF BEARING.
USE 3/4" RIVETS IN THE CONNECTION ANGLES.
THE WEIGHTS OF CONNECTION ANGLES INCLUDE THE HEADS OF SHOP RIVETS.

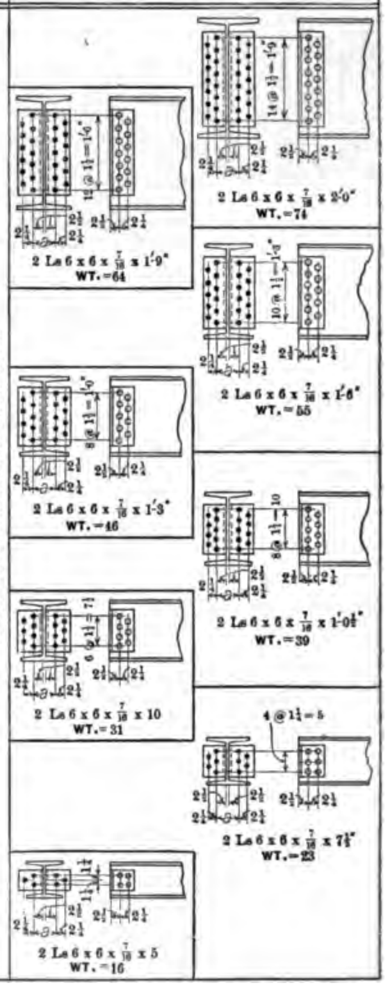
SIZE	WEIGHT	FLANGE	WEB		g	k	r	t	c	a	p
30	120	10 1/2	.54	9/16	6 1/2	1 3/4	1	15/16	5/16	5 9/16	TO BE DETERMINED
28	105	10	.50	1/2	6	1 5/8	1	7/8	5/16	5 1/2	
26	90	9 1/2	.46	1/2	5 1/2	1 1/2	1	3/4	5/16	5 1/2	
24	84	9 1/4	.46	1/2	5 1/4	1 1/2	7/8	13/16	5/16	5 1/2	16 x 3/4 x 1'4"
	83	9 1/8	.52	9/16	5 1/4	1 3/8	7/8	11/16	5/16	5 9/16	
	73	9	.39	3/8					1/4	5 3/8	
20	82	8 7/8	.57	9/16	5	1 7/16	7/8	3/4	5/16	5 9/16	16 x 3/4 x 1'3"
	72	8 3/4	.43	7/16					1/4	5 7/16	
	69	8 7/16	.52	9/16	4 1/2	1 1/4	7/8	5/8	5/16	5 9/16	
	64	8 1/16	.45	7/16					5/16	5 7/16	
	59	8	.38	3/8					1/4	5 3/8	
18	59	7 11/16	.50	1/2	4 1/4	1 1/8	7/8	9/16	5/16	5 1/2	16 x 3/4 x 1'2"
	54	7 5/8	.41	7/16					1/4	5 7/16	
	52	7 9/16	.38	3/8					1/4	5 3/8	
	48 1/2	7 1/2	.32	5/16					1/4	5 5/16	
15	71	7 1/2	.52	9/16	4 1/4	1 5/8	7/8	15/16	5/16	5 9/16	12 x 3/4 x 1'2"
	64	7 3/16	.61	5/8	4	1 3/8	7/8	3/4	3/8	5 5/8	
	54	7	.41	7/16					1/4	5 7/16	
	46	6 13/16	.44	7/16	3 3/4	1 1/16	7/8	1/2	5/16	5 7/16	
	41	6 3/4	.34	3/8					1/4	5 3/8	
	38	6 11/16	.29	5/16					3/16	5 5/16	
12	36	6 5/16	.31	5/16	3 1/2	1 1/16	3/4	7/16	3/16	5 5/16	12 x 3/4 x 1'0"
	32	6 1/4	.34	3/8	3 1/2	7/8	3/4	7/16	1/4	5 3/8	
	28 1/2	6 1/4	.25	1/4					1/8	5 1/4	
10	28 1/2	6	.39	3/8	3 1/4	13/16	3/4	3/8	1/4	5 3/8	10 x 3/4 x 10
	23 1/2	5 7/8	.25	1/4					3/16	5 1/4	
9	24	5 11/16	.37	3/8	3	3/4	3/4	3/8	1/4	5 3/8	8 x 3/4 x 9
	20	5 9/16	.25	1/4					3/16	5 1/4	
8	19 1/2	5 5/16	.33	5/16	2 3/4	11/16	3/4	5/16	1/4	5 5/16	8 x 3/4 x 8
	17 1/2	5 1/4	.25	1/4					3/16	5 1/4	

CONNECTION ANGLES



SIZE	WEIGHT	FLANGE	WEB		g	k	r	t	c	a	p
30	200	15	.75	3/4	11	2 3/8	1	1 1/8	7/16	5 3/4	TO BE DETERMINED FOR EACH BEAM
	180	13	.69	11/16	9	2 3/8	1	1 1/4	7/16	5 11/16	
28	180	14 3/8	.69	11/16	10 1/4	2 5/16	1	1 1/8	7/16	5 11/16	
	165	12 1/2	.66	11/16	8 1/2	2 5/16	1	1 3/16	3/8	5 11/16	
26	160	13 5/8	.63	5/8	9 1/2	2 3/16	1	1 1/16	3/8	5 5/8	
	150	12	.63	5/8	8	2 3/16	1	1 1/8	3/8	5 5/8	
24	140	13	.60	5/8	9	2	1	1	3/8	5 5/8	
	120	12	.53	9/16	8	1 7/8	1	15/16	5/16	5 9/16	
20	140	12 1/2	.64	5/8	8 1/2	2 1/8	1	1 1/8	3/8	5 5/8	
	112	12	.55	9/16	8	1 13/16	1	7/8	5/16	5 9/16	
18	92	11 1/2	.48	1/2	7 1/2	1 5/8	1	13/16	5/16	5 1/2	
15	140	11 3/4	.80	13/16	7 3/4	2 7/16	1	1 5/16	7/16	5 13/16	
	104	11 1/4	.60	5/8	7 1/4	1 15/16	1	15/16	3/8	5 5/8	
	73	10 1/2	.43	7/16	6 1/2	1 1/2	1	11/16	1/4	5 7/16	
12	70	10	.46	1/2	6	1 1/2	1	3/4	5/16	5 1/2	
	55	9 3/4	.37	3/8	6	1 1/4	1	5/8	1/4	5 3/8	
10	44	9	.31	5/16	5 1/2	1 1/8	7/8	9/16	3/16	5 5/16	
9	38	8 1/2	.30	5/16	5 1/4	1 1/16	7/8	1/2	3/16	5 5/16	
8	32 1/2	8	.29	5/16	5	1	7/8	7/16	3/16	5 5/16	

CONNECTION ANGLES



WEIGHTS, AREAS AND GAGES OF ANGLES

STANDARD

ADOPTED 1910 BY ASSOCIATION OF AMERICAN STEEL MANUFACTURERS

EQUAL LEGS

SIZE	THICKNESS	WEIGHT PER FT.	AREA
8 x 8	1 1/8	56.9	16.73
	1 1/16	54.0	15.87
	1	51.0	15.00
	15/16	48.1	14.12
	7/8	45.0	13.23
	13/16	42.0	12.34
	3/4	38.9	11.44
	11/16	35.8	10.53
	5/8	32.7	9.61
	9/16	29.6	8.68
	1/2	26.4	7.75
6 x 6	1	37.4	11.00
	15/16	35.3	10.37
	7/8	33.1	9.73
	13/16	31.0	9.09
	3/4	28.7	8.44
	11/16	26.5	7.78
	5/8	24.2	7.11
	9/16	21.9	6.43
	1/2	19.6	5.75
	7/16	17.2	5.06
	3/8	14.9	4.36
4 x 4	3/4	18.5	5.44
	11/16	17.1	5.03
	5/8	15.7	4.61
	9/16	14.3	4.18
	1/2	12.8	3.75
	7/16	11.3	3.31
	3/8	9.8	2.86
	5/16	8.2	2.40
3 1/2 x 3 1/2	5/8	13.6	3.98
	9/16	12.4	3.62
	1/2	11.1	3.25
	7/16	9.8	2.87
	3/8	8.5	2.48
	5/16	7.2	2.09
3 x 3	1/2	9.4	2.75
	7/16	8.3	2.43
	3/8	7.2	2.11
	5/16	6.1	1.78
	1/4	4.9	1.44
2 1/2 x 2 1/2	7/16	6.8	2.00
	3/8	5.9	1.73
	5/16	5.0	1.47
	1/4	4.1	1.19
	3/16	3.1	0.90
2 x 2	3/8	4.7	1.36
	5/16	3.9	1.15
	1/4	3.2	0.91
	3/16	2.4	0.71

UNEQUAL LEGS

SIZE	THICKNESS	WEIGHT PER FT.	AREA
6 x 4	7/8	27.2	7.98
	13/16	25.4	7.47
	3/4	23.6	6.94
	11/16	21.8	6.40
	5/8	20.0	5.86
	9/16	18.1	5.31
	1/2	16.2	4.75
	7/16	14.3	4.18
	3/8	12.3	3.61
6 x 3 1/2	7/8	25.7	7.55
	13/16	24.0	7.06
	3/4	22.4	6.56
	11/16	20.6	6.06
	5/8	18.9	5.55
	9/16	17.1	5.03
	1/2	15.3	4.50
	7/16	13.5	3.97
	3/8	11.7	3.42
5 x 3 1/2	3/4	19.8	5.81
	11/16	18.3	5.37
	5/8	16.8	4.92
	9/16	15.2	4.47
	1/2	13.6	4.00
	7/16	12.0	3.53
	3/8	10.4	3.05
	5/16	8.7	2.56
5 x 3	11/16	17.1	5.03
	5/8	15.7	4.61
	9/16	14.3	4.18
	1/2	12.8	3.75
	7/16	11.3	3.31
	3/8	9.8	2.86
	5/16	8.2	2.40
4 x 3	5/8	13.6	3.98
	9/16	12.4	3.62
	1/2	11.1	3.25
	7/16	9.8	2.87
	3/8	8.5	2.48
	5/16	7.2	2.09
3 1/2 x 3	9/16	11.4	3.34
	1/2	10.2	3.00
	7/16	9.1	2.65
	3/8	7.9	2.30
	5/16	6.6	1.93
3 1/2 x 2 1/2	1/2	9.4	2.75
	7/16	8.3	2.43
	3/8	7.2	2.11
	5/16	6.1	1.78
	1/4	4.9	1.44
3 x 2 1/2	7/16	7.6	2.21
	3/8	6.6	1.92
	5/16	5.6	1.62
	1/4	4.5	1.31
2 1/2 x 2	3/8	5.3	1.55
	5/16	4.5	1.31
	1/4	3.6	1.06
	3/16	2.8	0.81

GAGES

LEG	MAX. RIVET	g	g_1	g_2
8	1 1/8	4 1/2	3	3
7	1	4	2 1/2 [a]	3
6	1	3 1/2	2 1/2 [b]	2 1/4
5	1	3	2	1 3/4
4 1/2	1	2 1/2	2	1 1/4
4	1	2 1/2 [c]		
3 1/2	7/8	2 [d]		
3	7/8	1 3/4		
2 3/4	3/4	1 5/8		
2 1/2	3/4	1 3/8		
2 1/4	3/4	1 1/4		
2	5/8	1 1/8		

a — 2" FOR SOME COMPANIES.
b — 2 1/4" FOR SOME COMPANIES.
c — {2 1/4" FOR SOME COMPANIES, 2" IN BRACING}
d — 1 3/4" IN BRACING.

AREAS OF RIVET HOLES

THICKNESS OF METAL	DIAMETER OF HOLE (DIAMETER OF RIVET + 1/8") 5/8	3/4	7/8	1	1 1/8
2	1.25	1.50	1.75	2.00	2.25
1 15/16	1.21	1.45	1.70	1.94	2.18
1 7/8	1.17	1.41	1.64	1.88	2.11
1 13/16	1.13	1.36	1.59	1.81	2.04
1 3/4	1.09	1.31	1.53	1.75	1.97
1 11/16	1.05	1.27	1.47	1.69	1.90
1 5/8	1.02	1.22	1.42	1.63	1.83
1 9/16	.98	1.17	1.37	1.56	1.76
1 1/2	.94	1.13	1.31	1.50	1.69
1 7/16	.90	1.08	1.26	1.44	1.62
1 3/8	.86	1.03	1.20	1.38	1.55
1 5/16	.82	.98	1.15	1.31	1.48
1 1/4	.78	.94	1.09	1.25	1.41
1 3/16	.74	.89	1.04	1.19	1.34
1 1/8	.70	.84	.98	1.13	1.27
1 1/16	.66	.80	.93	1.06	1.20
1	.63	.75	.88	1.00	1.13
15/16	.59	.70	.82	.94	1.05
7/8	.55	.66	.77	.88	.98
13/16	.51	.61	.71	.81	.91
3/4	.47	.56	.66	.75	.84
11/16	.43	.52	.60	.69	.77
5/8	.39	.47	.55	.63	.70
9/16	.35	.42	.49	.56	.63
1/2	.31	.38	.44	.50	.56
7/16	.27	.33	.38	.44	.49
3/8	.23	.28	.33	.38	.42
5/16	.20	.23	.27	.31	.35
1/4	.16	.19	.22	.25	.28
3/16	.12	.14	.16	.19	.21

SPECIAL (SEE NOTE BELOW)

*NOT ROLLED BY ALL THE LEADING STEEL COMPANIES † ROLLED ONLY BY ONE OR TWO COMPANIES

EQUAL LEGS

SIZE	THICKNESS	WEIGHT PER FT.	AREA
5 x 5	* 1	30.6	9.00
	* 15/16	28.9	8.50
	* 7/8	27.2	7.98
	* 13/16	25.4	7.46
	* 3/4	23.6	6.94
	11/16	21.8	6.40
	5/8	20.0	5.86
	9/16	18.1	5.31
	1/2	16.2	4.75
	7/16	14.3	4.18
	3/8	12.3	3.61
	† 5/16	10.3	3.03
4 x 4	† 7/8	21.2	6.23
	13/16	19.9	5.84
	* 1/4	6.6	1.94
3 1/2 x 3 1/2	† 7/8	18.3	5.36
	13/16	17.1	5.03
	3/4	16.0	4.69
	11/16	14.8	4.31
	* 1/4	5.8	1.69
	† 5/32	3.6	1.07
3 1/4 x 3 1/4	† 3/4	14.7	4.31
	† 11/16	13.6	4.00
	† 5/8	12.5	3.67
	† 9/16	11.4	3.31
	† 1/2	10.2	3.00
	† 7/16	9.1	2.65
	† 3/8	7.9	2.30
	† 5/16	6.6	1.93
	† 1/4	5.4	1.56
	† 5/32	3.4	0.99
3 x 3	* 5/8	11.5	3.36
	9/16	10.4	3.06
	† 3/16	3.7	1.09
	† 1/8	2.5	0.73
2 3/4 x 2 3/4	† 1/2	8.5	2.50
	† 7/16	7.6	2.21
	† 3/8	6.6	1.92
	† 5/16	5.6	1.62
	† 1/4	4.5	1.31
	† 3/16	3.4	1.00
	† 1/8	2.3	0.67
2 1/2 x 2 1/2	1/2	7.7	2.25
	† 1/8	2.1	0.61
2 1/4 x 2 1/4	* 1/2	6.8	2.00
	* 7/16	6.1	1.78
	* 3/8	5.3	1.55
	* 5/16	4.5	1.31
	* 1/4	3.6	1.06
	* 3/16	2.8	0.81
	† 1/8	1.9	0.55
2 x 2	† 1/2	6.0	1.75
	7/16	5.3	1.56
	† 1/8	1.7	0.48

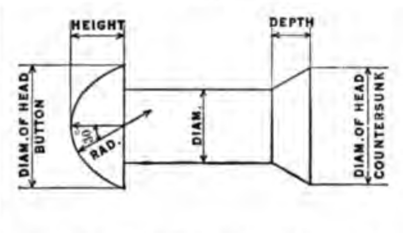
UNEQUAL LEGS

SIZE	THICKNESS	WEIGHT PER FT.	AREA
8 x 6	* 1 1/8	49.3	14.49
	* 1 1/16	46.8	13.75
	1	44.2	13.00
	15/16	41.7	12.25
	7/8	39.1	11.48
	13/16	36.5	10.71
	3/4	33.8	9.94
	11/16	31.2	9.15
	5/8	28.5	8.36
	9/16	25.7	7.56
	1/2	23.0	6.75
	† 7/16	20.2	5.93
8 x 3 1/2	† 1	35.7	10.50
	† 15/16	33.7	9.90
	† 7/8	31.7	9.30
	† 13/16	29.6	8.68
	† 3/4	27.5	8.06
	† 11/16	25.3	7.43
	† 5/8	23.2	6.80
	† 9/16	21.0	6.15
	† 1/2	18.7	5.50
	† 7/16	16.5	4.84
7 x 3 1/2	* 1	32.3	9.50
	* 15/16	30.5	8.97
	7/8	28.7	8.42
	13/16	26.8	7.87
	3/4	24.9	7.31
	11/16	23.0	6.75
	5/8	21.0	6.17
	9/16	19.1	5.59
	1/2	17.0	5.00
	7/16	15.0	4.40
	* 3/8	13.0	3.80
6 x 4*	1	30.6	9.00
	15/16	28.9	8.50
6 x 3 1/2*	1	28.9	8.50
	15/16	27.3	8.03
	† 5/16	9.8	2.87
5 x 4	* 7/8	24.2	7.11
	* 13/16	22.7	6.65
	* 3/4	21.1	6.19
	11/16	19.5	5.72
	5/8	17.8	5.23
	9/16	16.2	4.75
	1/2	14.5	4.25
	7/16	12.8	3.75
	3/8	11.0	3.23
	† 5/16	9.3	2.71
5 x 3 1/2	† 15/16	24.2	7.09
	* 7/8	22.7	6.67
	* 13/16	21.3	6.25
5 x 3	* 7/8	21.2	6.23
	* 13/16	19.9	5.84
	3/4	18.5	5.44
	† 1/4	6.6	1.94
4 1/2 x 3	* 13/16	18.5	5.43
	* 3/4	17.3	5.06
	* 11/16	16.0	4.68
	* 5/8	14.7	4.30
	* 9/16	13.3	3.90
	* 1/2	11.9	3.50
	* 7/16	10.6	3.09
	* 3/8	9.1	2.67
	* 5/16	7.7	2.25
	† 1/4	6.2	1.81
4 x 3 1/2	† 7/8	19.8	5.80
	* 13/16	18.5	5.43
	* 3/4	17.3	5.06
	* 11/16	16.0	4.68
	5/8	14.7	4.30
	9/16	13.3	3.90
	1/2	11.9	3.50
	7/16	10.6	3.09
	3/8	9.1	2.67
	5/16	7.7	2.25
	† 1/4	6.2	1.81
4 x 3*	† 7/8	18.3	5.36
	13/16	17.1	5.03
	3/4	16.0	4.69
	11/16	14.8	4.34
	† 1/4	5.8	1.69
3 1/2 x 3	† 7/8	16.8	4.92
	* 13/16	15.8	4.62
	* 3/4	14.7	4.31
	* 11/16	13.6	4.00
	5/8	12.5	3.67
	* 1/4	5.4	1.56
3 1/2 x 2 1/2	† 3/4	13.4	3.94
	* 11/16	12.5	3.65
	5/8	11.5	3.36
	9/16	10.4	3.06
3 x 2 1/2†	5/8	10.4	3.05
	9/16	9.5	2.78
	1/2	8.5	2.50
	† 3/16	3.4	1.00
3 x 2	† 9/16	8.5	2.50
	1/2	7.7	2.25
	7/16	6.8	2.00
	3/8	5.9	1.73
	5/16	5.0	1.47
	1/4	4.1	1.19
	* 3/16	3.1	0.90
2 1/2 x 2	* 1/2	6.8	2.00
	* 7/16	6.1	1.78
	† 1/8	1.9	0.55

NOTE:- BASED UPON STANDARDS OF THE CARNEGIE, CAMBRIA, LACKAWANNA, JONES AND LAUGHLIN, AND PENNSYLVANIA STEEL COMPANIES. PHŒNIX SECTIONS ARE SIMILAR.

RIVETS AND BOLTS

WEIGHTS AND DIMENSIONS OF RIVETS



RIVETS SHOULD NOT BE COUNTERSUNK IN PLATES THINNER THAN THOSE GIVEN IN TABLE

SHANK		BUTTON HEAD				C'S'K HEAD		
DIAM.	WEIGHT 100 IN.	HEIGHT	DIAM.	RADIUS	WEIGHT PER 100	DEPTH	DIAM.	MIN. PLATE
$\frac{1}{2}$	5.6	$\frac{3}{8}$	$\frac{7}{8}$	$\frac{9}{16}$	5.0	$\frac{1}{4}$	$\frac{13}{16}$	$\frac{1}{4}$
$\frac{5}{8}$	8.7	$\frac{7}{16}$	$1\frac{1}{16}$	$\frac{11}{16}$	9.7	$\frac{5}{16}$	1	$\frac{5}{16}$
$\frac{3}{4}$	12.5	$\frac{9}{16}$	$1\frac{1}{4}$	$\frac{13}{16}$	16.0	$\frac{3}{8}$	$1\frac{3}{16}$	$\frac{3}{8}$
$\frac{7}{8}$	17.0	$\frac{5}{8}$	$1\frac{7}{16}$	$\frac{15}{16}$	24.0	$\frac{7}{16}$	$1\frac{3}{8}$	$\frac{7}{16}$
1	22.3	$\frac{11}{16}$	$1\frac{5}{8}$	$1\frac{1}{16}$	35.0	$\frac{1}{2}$	$1\frac{9}{16}$	$\frac{1}{2}$

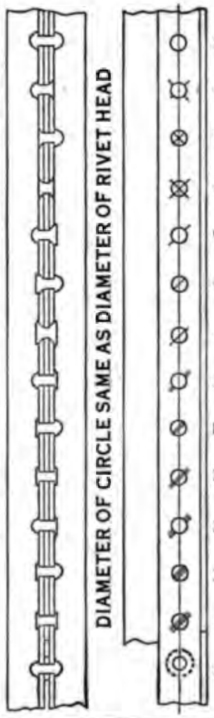
WEIGHTS AND DIMENSIONS OF BOLTS

MANUFACTURERS STANDARD

SHANK		HEXAGONAL HEAD				HEXAGONAL NUT				SQUARE HEAD				SQUARE NUT			
DIAM.	WEIGHT 100 IN.	HEIGHT	SHORT DIAM.	LONG DIAM.	WEIGHT PER 100	HEIGHT	SHORT DIAM.	LONG DIAM.	WEIGHT PER 100	HEIGHT	SHORT DIAM.	LONG DIAM.	WEIGHT PER 100	HEIGHT	SHORT DIAM.	LONG DIAM.	WEIGHT PER 100
$\frac{1}{2}$	5.6	$\frac{3}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	5.1	$\frac{1}{2}$	1	$1\frac{3}{16}$	9.9	$\frac{3}{8}$	$\frac{3}{4}$	$1\frac{1}{16}$	5.9	$\frac{1}{2}$	1	$1\frac{7}{16}$	11.9
$\frac{5}{8}$	8.7	$\frac{1}{2}$	$\frac{15}{16}$	$1\frac{1}{8}$	10.0	$\frac{5}{8}$	$1\frac{1}{4}$	$1\frac{7}{16}$	19.5	$\frac{1}{2}$	$\frac{15}{16}$	$1\frac{5}{16}$	11.5	$\frac{5}{8}$	$1\frac{1}{4}$	$1\frac{13}{16}$	23.0
$\frac{3}{4}$	12.5	$\frac{9}{16}$	$1\frac{1}{8}$	$1\frac{5}{16}$	17.3	$\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	34.5	$\frac{9}{16}$	$1\frac{1}{8}$	$1\frac{5}{8}$	19.9	$\frac{3}{4}$	$1\frac{1}{2}$	$2\frac{1}{8}$	41.0
$\frac{7}{8}$	17.0	$\frac{11}{16}$	$1\frac{5}{16}$	$1\frac{1}{2}$	27.4	$\frac{7}{8}$	$1\frac{5}{8}$	$1\frac{7}{8}$	45.3	$\frac{11}{16}$	$1\frac{5}{16}$	$1\frac{7}{8}$	31.1	$\frac{7}{8}$	$1\frac{3}{4}$	$2\frac{1}{2}$	61.3
1	22.3	$\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	42.0	1	$1\frac{3}{4}$	$2\frac{1}{16}$	57.5	$\frac{3}{4}$	$1\frac{1}{2}$	$2\frac{1}{8}$	47.3	1	2	$2\frac{13}{16}$	95.2

RIVET CODE

SHOP RIVETS



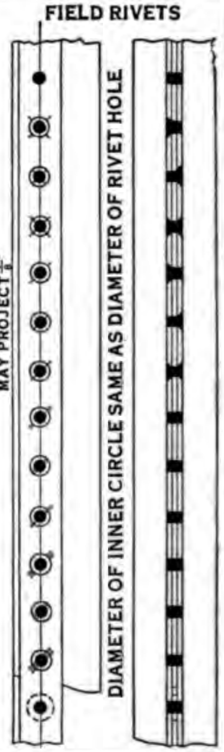
EXPLANATION

- FULL BUTTON HEADS, BOTH SIDES
- COUNTERSUNK AND CHIPPED, NEAR SIDE
- COUNTERSUNK AND CHIPPED, FAR SIDE
- COUNTERSUNK AND CHIPPED, BOTH SIDES
- COUNTERSUNK BUT NOT CHIPPED, NEAR SIDE
- COUNTERSUNK BUT NOT CHIPPED, FAR SIDE
- COUNTERSUNK BUT NOT CHIPPED, BOTH SIDES

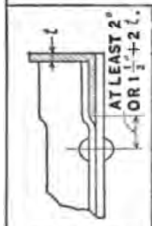
MAY PROJECT $\frac{1}{8}''$

- FLATTENED TO $\frac{1}{4}''$ HIGH, NEAR SIDE
- FLATTENED TO $\frac{1}{4}''$ HIGH, FAR SIDE
- FLATTENED TO $\frac{1}{4}''$ HIGH, BOTH SIDES
- FLATTENED TO $\frac{3}{8}''$ HIGH, NEAR SIDE
- FLATTENED TO $\frac{3}{8}''$ HIGH, FAR SIDE
- FLATTENED TO $\frac{3}{8}''$ HIGH, BOTH SIDES
- STITCH RIVET WITH WASHER

FIELD RIVETS

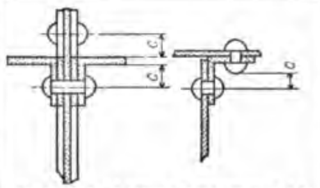


CLEARANCE FOR MACHINE DRIVEN RIVETS



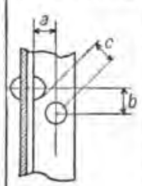
CRIMPED ANGLE

DIAM.	c
$\frac{1}{2}$	1
$\frac{5}{8}$	$1\frac{1}{8}$
$\frac{3}{4}$	$1\frac{1}{4}$
$\frac{7}{8}$	$1\frac{3}{8}$
1	$1\frac{1}{2}$



USUAL VALUES OF "c" GIVEN IN TABLE. IN EXTREME CASES "c" MAY BE TAKEN $\frac{3}{8}''$ + ONE HALF DIAM. OF HEAD.

MINIMUM RIVET STAGGER "b" FOR USUAL VALUES OF "c" AND THE FOLLOWING VALUES OF "a"



DIAM.	1	$1\frac{1}{16}$	$1\frac{1}{8}$	$1\frac{3}{16}$	$1\frac{1}{4}$	$1\frac{5}{16}$	$1\frac{3}{8}$	$1\frac{7}{16}$	$1\frac{1}{2}$	$1\frac{9}{16}$	$1\frac{5}{8}$	$1\frac{11}{16}$	$1\frac{3}{4}$	$1\frac{13}{16}$	$1\frac{7}{8}$	$1\frac{15}{16}$	2	$2\frac{1}{16}$	$2\frac{1}{8}$	$2\frac{3}{16}$
$\frac{1}{2}$	$\frac{15}{16}$	$\frac{7}{8}$	$\frac{13}{16}$	$\frac{11}{16}$	$\frac{9}{16}$	$\frac{7}{16}$	0													
$\frac{5}{8}$			$1\frac{1}{8}$	$1\frac{1}{16}$	$\frac{15}{16}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{1}{4}$	0									
$\frac{3}{4}$					$1\frac{1}{4}$	$1\frac{3}{16}$	$1\frac{1}{8}$	$1\frac{1}{16}$	$\frac{15}{16}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{9}{16}$	$\frac{5}{16}$	0						
$\frac{7}{8}$							$1\frac{7}{16}$	$1\frac{3}{8}$	$1\frac{5}{16}$	$1\frac{1}{4}$	$1\frac{1}{8}$	$1\frac{1}{16}$	$\frac{15}{16}$	$\frac{13}{16}$	$\frac{11}{16}$	$\frac{7}{16}$	0			
1									$1\frac{5}{8}$	$1\frac{9}{16}$	$1\frac{1}{2}$	$1\frac{7}{16}$	$1\frac{5}{16}$	$1\frac{1}{4}$	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{9}{16}$	0

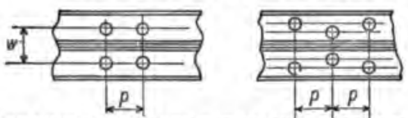
RIVET SPACING*

MINIMUM RIVET SPACING "m"

SEE ALSO FOLLOWING PAGE.

DIAMETER	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
THREE DIAMETERS	$1\frac{1}{8}$	$1\frac{1}{2}$	$1\frac{7}{8}$	$2\frac{1}{4}$	$2\frac{5}{8}$	3
USUAL MINIMUM	$1\frac{1}{4}$	$1\frac{3}{4}$	2	$2\frac{1}{4}$	$2\frac{5}{8}$	3
PREFERRED	$1\frac{1}{4}$	$1\frac{3}{4}$	$2\frac{1}{4}$	$2\frac{1}{2}$	3	$3\frac{1}{4}$

MAXIMUM RIVET SPACING

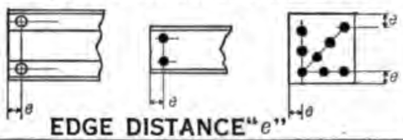


THE RIVET PITCH "p" OR THE SPACING MEASURED PARALLEL TO THE LINE OF STRESS, SHOULD NOT EXCEED 6" OR 16 t FOR $\frac{7}{8}$" OR $\frac{3}{4}$" RIVETS, OR $4\frac{1}{2}$" OR 16 t FOR $\frac{5}{8}$" RIVETS, WHERE t IS THE THICKNESS OF THE THINNEST OUTSIDE PLATE.

IN GIRDERS OR STRINGERS WHICH SUPPORT MOVING LOADS APPLIED TO THE FLANGES, A MAXIMUM PITCH OF 4" OR $4\frac{1}{2}$" IS OFTEN SPECIFIED.

THE MAXIMUM PITCH AT THE ENDS OF COMPRESSION MEMBERS IS FOUR DIAMETERS FOR A DISTANCE OF ONE AND ONE HALF TIMES THE WIDTH OF THE MEMBER.

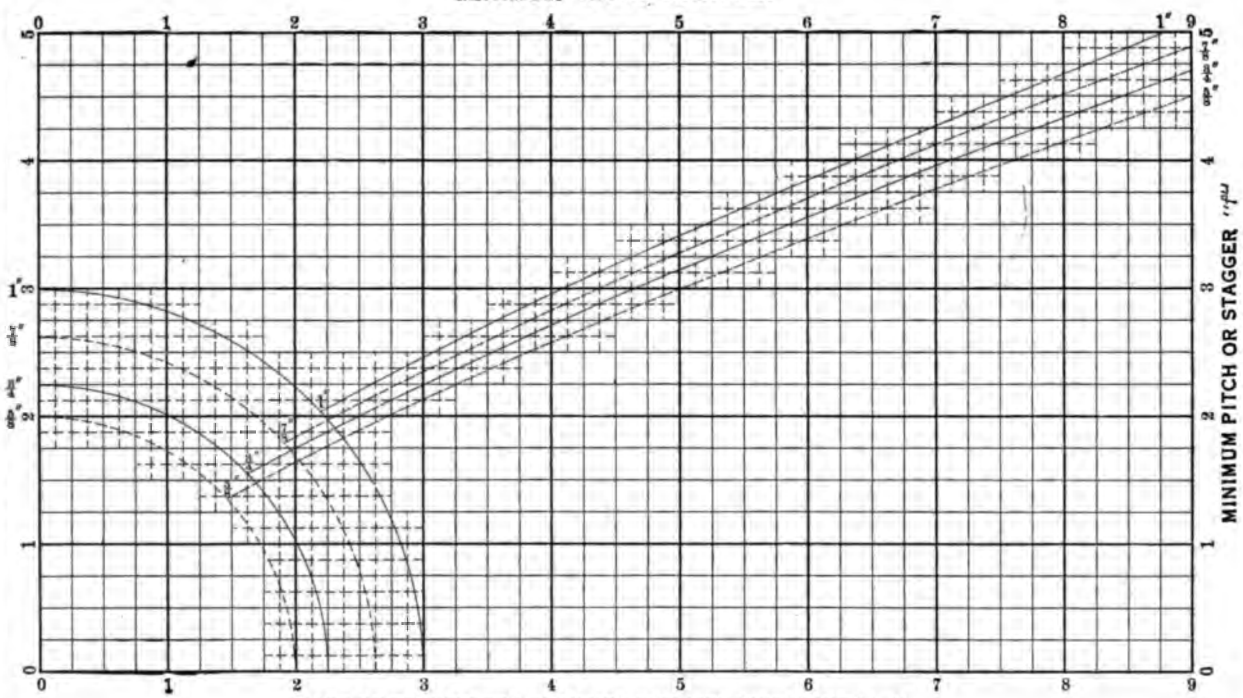
THE MAXIMUM RIVET SPACING "w" MEASURED PERPENDICULARLY TO THE LINE OF STRESS IS 40 t.



EDGE DISTANCE "e"

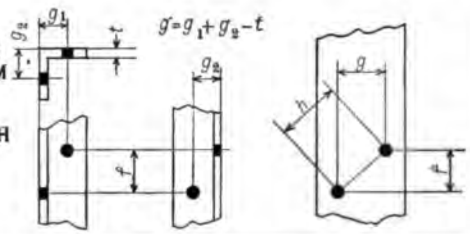
DIAMETER	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
EXTREME MINIMUM	$\frac{9}{16}$	$\frac{3}{4}$	$\frac{15}{16}$	$1\frac{1}{8}$	$1\frac{5}{16}$	$1\frac{1}{2}$
USUAL MINIMUM	$\frac{3}{4}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$
PREFERRED	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2
MAXIMUM	5" OR 8 t (SEE ABOVE)					

MINIMUM RIVET STAGGER



USE THE CIRCULAR ARCS TO DETERMINE THE MINIMUM STAGGER (f) WHICH WILL MAKE THE DISTANCE C. TO C. HOLES, (h) EQUAL TO THE USUAL MINIMUM SPACING (m) GIVEN ABOVE.

USE THE OTHER CURVES TO DETERMINE THE MINIMUM STAGGER (f) WHICH CAN BE USED IN A TENSION MEMBER WITHOUT REDUCING THE AREA OF CROSS SECTION BY THE AREA OF AN ADDITIONAL RIVET HOLE



* ON THIS PAGE ALL DIAMETERS ARE RIVET DIAMETERS AND ALL DIMENSIONS ARE IN INCHES.

MINIMUM PITCHES FOR FLANGE RIVETS

RIVETS IN A SINGLE LINE

UNIT STRESSES, POUNDS PER SQUARE INCH	d \ t	Single shear 5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	Double shear 1/2 or under	9/16	5/8	11/16	3/4
SHEAR ON WEB (NET SECTION) s = 13,000; SHEAR ON RIVETS s' = 12,000; BEARING IN WEB b = 24,000	5/8	1 11/16	1 1/2	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 3/4	1 3/4	1 11/16	1 9/16	1 1/2
	3/4	2 3/16	2	1 13/16	1 11/16	1 5/8	1 5/8	1 5/8	1 5/8	2 1/4	2 1/4	2 3/16	2 1/16	2
	7/8	2 5/8	2 3/8	2 1/4	2 1/8	2	1 7/8	1 13/16	1 13/16	2 5/8	2 5/8	2 1/2	2 3/8	2 1/4
	1	3	3	2 13/16	2 9/16	2 7/16	2 5/16	2 3/16	2 1/16	3	3	3	3	3
	1 1/8	3 7/16	3 5/16	3 3/16	3 1/8	2 7/8	2 3/4	2 5/8	2 1/2	3 7/16	3 7/16	3 7/16	3 7/16	3 7/16
	1 1/4	3 11/16	3 11/16	3 11/16	3 5/8	3 3/8	3 3/16	3 1/16	2 7/8	3 11/16	3 11/16	3 11/16	3 11/16	3 11/16
SHEAR ON WEB (NET SECTION) s = 13,000; SHEAR ON RIVETS s' = 11,000; BEARING IN WEB b = 22,000	5/8	1 9/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 13/16	1 11/16	1 9/16	1 1/2	1 7/16
	3/4	2 1/16	1 7/8	1 3/4	1 5/8	1 5/8	1 5/8	1 5/8	1 5/8	2 1/8	2 1/8	2 1/16	2	1 7/8
	7/8	2 1/2	2 3/8	2 3/16	2	1 13/16	1 13/16	1 13/16	1 13/16	2 1/2	2 1/2	2 1/2	2 1/2	2 3/8
	1	2 15/16	2 15/16	2 5/8	2 7/16	2 5/16	2 3/16	2 1/8	2	2 15/16	2 15/16	2 15/16	2 15/16	2 15/16
	1 1/8	3 3/8	3 3/8	3 3/8	2 15/16	2 3/4	2 5/8	2 1/2	2 3/8	3 3/8	3 3/8	3 3/8	3 3/8	3 3/8
	1 1/4	3 1/2	3 1/2	3 1/2	3 7/16	3 1/4	3 1/16	2 7/8	2 3/4	3 1/2	3 1/2	3 1/2	3 1/2	3 1/2
SHEAR ON WEB (NET SECTION) s = 12,000; SHEAR ON RIVETS s' = 12,000; BEARING IN WEB b = 24,000	5/8	1 3/4	1 9/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	2	1 7/8	1 3/4	1 5/8	1 9/16
	3/4	2 5/16	2 1/16	1 7/8	1 3/4	1 11/16	1 5/8	1 5/8	1 5/8	2 3/8	2 3/8	2 5/16	2 3/16	2 1/16
	7/8	2 3/4	2 5/8	2 1/2	2 5/16	2 1/16	2	1 7/8	1 13/16	2 3/4	2 3/4	2 3/4	2 3/4	2 5/8
	1	3 1/8	3 1/8	2 15/16	2 11/16	2 9/16	2 3/8	2 1/4	2 3/16	3 1/8	3 1/8	3 1/8	3 1/8	3 1/8
	1 1/8	3 1/2	3 1/2	3 1/2	3 1/4	3	2 7/8	2 13/16	2 9/16	3 1/2	3 1/2	3 1/2	3 1/2	3 1/2
	1 1/4	3 7/8	3 7/8	3 7/8	3 13/16	3 9/16	3 3/8	3 3/16	3	3 7/8	3 7/8	3 7/8	3 7/8	3 7/8
SHEAR ON WEB (NET SECTION) s = 12,000; SHEAR ON RIVETS s' = 11,000; BEARING IN WEB b = 22,000	5/8	1 11/16	1 1/2	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/8	1 3/4	1 11/16	1 9/16	1 1/2
	3/4	2 3/16	1 15/16	1 13/16	1 11/16	1 5/8	1 5/8	1 5/8	1 5/8	2 1/4	2 1/4	2 3/16	2 1/16	1 15/16
	7/8	2 5/8	2 3/8	2 1/4	2 1/8	2	1 7/8	1 13/16	1 13/16	2 5/8	2 5/8	2 5/8	2 5/8	2 1/2
	1	2 15/16	2 15/16	2 13/16	2 9/16	2 7/16	2 5/16	2 3/16	2	2 15/16	2 15/16	2 15/16	2 15/16	2 15/16
	1 1/8	3 5/16	3 5/16	3 5/16	3 1/16	2 7/8	2 3/4	2 9/16	2 1/2	3 5/16	3 5/16	3 5/16	3 5/16	3 5/16
	1 1/4	3 11/16	3 11/16	3 11/16	3 5/8	3 3/8	3 3/16	3	2 7/8	3 11/16	3 11/16	3 11/16	3 11/16	3 11/16
SHEAR ON WEB (NET SECTION) s = 12,000; SHEAR ON RIVETS s' = 10,000; BEARING IN WEB b = 20,000	5/8	1 9/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 3/4	1 11/16	1 9/16	1 1/2	1 7/16
	3/4	2 1/16	1 7/8	1 3/4	1 5/8	1 5/8	1 5/8	1 5/8	1 5/8	2 1/8	2 1/8	2 1/16	1 15/16	1 7/8
	7/8	2 7/16	2 3/8	2 3/16	2	1 7/8	1 13/16	1 13/16	1 13/16	2 7/16	2 7/16	2 7/16	2 7/16	2 3/8
	1	2 13/16	2 13/16	2 5/8	2 7/16	2 5/16	2 3/16	2 1/16	2	2 13/16	2 13/16	2 13/16	2 13/16	2 13/16
	1 1/8	3 1/8	3 1/8	3 1/8	2 15/16	2 3/4	2 5/8	2 7/16	2 3/8	3 1/8	3 1/8	3 1/8	3 1/8	3 1/8
	1 1/4	3 7/16	3 7/16	3 7/16	3 7/16	3 5/16	3	2 7/8	2 3/4	3 7/16	3 7/16	3 7/16	3 7/16	3 7/16

RIVETS STAGGERED IN TWO LINES

UNIT STRESSES, POUNDS PER SQUARE INCH	d \ t	Single shear 5/16	3/8	7/16	1/2	9/16	5/8	11/16	Double shear 1/2 or under	9/16	5/8	11/16	3/4
s = 13,000; s' = 12,000; b = 24,000	5/8	1 5/16	1 1/4	1 3/16	1 3/16	1 3/16	1 3/16	1 3/16	1 1/2	1 3/8	1 5/16	1 5/16	1 1/4
	3/4	1 5/8	1 9/16	1 7/16	1 7/16	1 7/16	1 7/16	1 7/16	1 13/16	1 13/16	1 3/4	1 5/8	1 9/16
	7/8	2 1/8	2	1 3/4	1 5/8	1 1/2	1 1/2	1 1/2	2 1/8	2 1/8	2 1/8	2 1/8	2
	1	2 7/16	2 7/16	2 1/4	2	1 7/8	1 3/4	1 5/8	2 7/16	2 7/16	2 7/16	2 7/16	2 7/16
	1 1/8	2 13/16	2 13/16	2 13/16	2 1/2	2 1/4	2 1/8	2	2 13/16	2 13/16	2 13/16	2 13/16	2 13/16
	1 1/4	3	3	3	2 13/16	2 11/16	2 1/2	2 3/8	3	3	3	3	3
s = 13,000; s' = 11,000; b = 22,000	5/8	1 5/16	1 3/16	1 3/16	1 3/16	1 3/16	1 3/16	1 3/16	1 7/16	1 5/16	1 5/16	1 1/4	1 3/16
	3/4	1 5/8	1 7/16	1 5/16	1 1/4	1 1/4	1 1/4	1 1/4	1 11/16	1 11/16	1 5/8	1 9/16	1 7/16
	7/8	2	1 7/8	1 13/16	1 5/8	1 9/16	1 1/2	1 1/2	2	2	2	2	1 7/8
	1	2 1/4	2 1/4	2 1/16	1 7/8	1 3/4	1 5/8	1 9/16	2 1/4	2 1/4	2 1/4	2 1/4	2 1/4
	1 1/8	2 1/2	2 1/2	2 1/2	2 5/16	2 1/8	1 7/8	1 3/4	2 1/2	2 1/2	2 1/2	2 1/2	2 1/2
	1 1/4	2 13/16	2 13/16	2 13/16	2 3/4	2 9/16	2 3/8	2 3/16	2 13/16	2 13/16	2 13/16	2 13/16	2 13/16
s = 12,000; s' = 12,000; b = 24,000	5/8	1 5/8	1 5/16	1 3/16	1 3/16	1 3/16	1 3/16	1 3/16	1 5/8	1 5/8	1 5/8	1 1/2	1 5/16
	3/4	1 7/8	1 5/8	1 7/16	1 5/16	1 1/4	1 1/4	1 1/4	1 15/16	1 15/16	1 7/8	1 3/4	1 5/8
	7/8	2 1/4	2 1/8	1 3/4	1 13/16	1 9/16	1 1/2	1 1/2	2 1/4	2 1/4	2 1/4	2 1/4	2 1/8
	1	2 9/16	2 9/16	2 3/8	2 1/8	2	1 13/16	1 11/16	2 9/16	2 9/16	2 9/16	2 9/16	2 9/16
	1 1/8	2 7/8	2 7/8	2 7/8	2 5/8	2 3/8	2 1/4	2 1/16	2 7/8	2 7/8	2 7/8	2 7/8	2 7/8
	1 1/4	3 3/16	3 3/16	3 3/16	3 1/8	2 7/8	2 11/16	2 1/2	3 3/16	3 3/16	3 3/16	3 3/16	3 3/16
s = 12,000; s' = 11,000; b = 22,000	5/8	1 5/16	1 1/4	1 3/16	1 3/16	1 3/16	1 3/16	1 3/16	1 1/2	1 3/8	1 5/16	1 5/16	1 1/4
	3/4	1 5/8	1 1/2	1 3/8	1 1/4	1 1/4	1 1/4	1 1/4	1 13/16	1 13/16	1 3/4	1 5/8	1 1/2
	7/8	2 1/8	2	1 3/4	1 5/8	1 1/2	1 1/2	1 1/2	2 1/8	2 1/8	2 1/8	2 1/8	2
	1	2 3/8	2 3/8	2 1/4	2	1 7/8	1 3/4	1 5/8	2 3/8	2 3/8	2 3/8	2 3/8	2 3/8
	1 1/8	2 11/16	2 11/16	2 11/16	2 7/16	2 1/4	2 1/8	1 15/16	2 11/16	2 11/16	2 11/16	2 11/16	2 11/16
	1 1/4	3	3	3	2 13/16	2 11/16	2 1/2	2 5/16	3	3	3	3	3
s = 12,000; s' = 10,000; b = 20,000	5/8	1 5/16	1 3/16	1 3/16	1 3/16	1 3/16	1 3/16	1 3/16	1 3/8	1 5/16	1 5/16	1 1/4	1 3/16
	3/4	1 1/2	1 7/16	1 5/16	1 1/4	1 1/4	1 1/4	1 1/4	1 11/16	1 11/16	1 5/8	1 1/2	1 7/16
	7/8	1 13/16	1 7/8	1 13/16	1 1/2	1 1/2	1 1/2	1 1/2	1 13/16	1 13/16	1 13/16	1 13/16	1 3/4
	1	2 1/4	2 1/4	2 1/16	1 7/8	1 3/4	1 5/8	1 1/2	2 1/4	2 1/4	2 1/4	2 1/4	2 1/4
	1 1/8	2 1/2	2 1/2	2 1/2	2 5/16	2 1/8	1 15/16	1 15/16	2 1/2	2 1/2	2 1/2	2 1/2	2 1/2
	1 1/4	2 3/4	2 3/4	2 3/4	2 3/4	2 1/2	2 5/16	2 5/16	2 3/4	2 3/4	2 3/4	2 3/4	2 3/4

NO VALUE GIVEN IS LESS THAN THE MINIMUM SPACING FOR MACHINE DRIVEN RIVETS, EQUAL TO THE DIAMETER OF THE RIVET HEAD PLUS 1/8".
VALUES ABOVE THE SHORT FINE LINES ARE LESS THAN THE MINIMUM STAGGER FOUND FROM THE DIAGRAM ON THE PRECEDING PAGE FOR GAGE $g = 2\frac{1}{2}''$.
FOR THE MINIMUM STAGGER FOR OTHER GAGES, SEE DIAGRAM.

PITCH OF RIVETS IN INCHES

SPACES	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½	2⅝	2¾	2⅞	3	3⅛	3¼	3⅜	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6	SPACES
2	2¼	2½	2¾	3	3¼	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6	6¼	6½	6¾	7	7½	8	8½	9	9½	10	10½	11	11½	1-0	**2**
3	3⅜	3¾	4⅛	4½	4⅞	5¼	5⅝	6	6⅜	6¾	7⅛	7½	7⅞	8¼	8⅝	9	9⅜	9¾	10⅛	10½	11¼	1-0	1-0¾	1-1½	1-2¼	1-3	1-3¾	1-4½	1-5¼	1-6	**3**
4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	1-0	1-0½	1-1	1-1½	1-2	1-3	1-4	1-5	1-6	1-7	1-8	1-9	1-10	1-11	2-0	**4**
5	5⅝	6¼	6⅞	7½	8⅛	8¾	9⅜	10	10⅝	11¼	11⅞	1-0½	1-1⅛	1-1¾	1-2⅜	1-3	1-3⅝	1-4¼	1-4⅞	1-5½	1-6¾	1-8	1-9¼	1-10½	1-11¾	2-1	2-2¼	2-3½	2-4¾	2-6	**5**
6	6¾	7½	8¼	9	9¾	10½	11¼	1-0	1-0¾	1-1½	1-2¼	1-3	1-3¾	1-4½	1-5¼	1-6	1-6¾	1-7½	1-8¼	1-9	1-10½	2-0	2-1½	2-3	2-4½	2-6	2-7½	2-9	2-10½	3-0	**6**
7	7⅞	8¾	9⅝	10½	11⅜	1-0¼	1-1⅛	1-2	1-2⅞	1-3¾	1-4⅝	1-5½	1-6⅜	1-7¼	1-8⅛	1-9	1-9⅞	1-10¾	1-11⅝	2-0½	2-2¼	2-4	2-5¾	2-7½	2-9¼	2-11	3-0¾	3-2½	3-4¼	3-6	**7**
8	9	10	11	1-0	1-1	1-2	1-3	1-4	1-5	1-6	1-7	1-8	1-9	1-10	1-11	2-0	2-1	2-2	2-3	2-4	2-6	2-8	2-10	3-0	3-2	3-4	3-6	3-8	3-10	4-0	**8**
9	10⅛	11¼	1-0⅜	1-1½	1-2⅝	1-3¾	1-4⅞	1-6	1-7⅛	1-8¼	1-9⅜	1-10½	1-11⅝	2-0¾	2-1⅞	2-3	2-4⅛	2-5¼	2-6⅜	2-7½	2-9¾	3-0	3-2¼	3-4½	3-6¾	3-9	3-11¼	4-1½	4-3¾	4-6	**9**
10	11¼	1-0½	1-1¾	1-3	1-4¼	1-5½	1-6¾	1-8	1-9¼	1-10½	1-11¾	2-1	2-2¼	2-3½	2-4¾	2-6	2-7¼	2-8½	2-9¾	2-11	3-1½	3-4	3-6½	3-9	3-11½	4-2	4-4½	4-7	4-9½	5-0	**10**
11	1-0⅜	1-1¾	1-3⅛	1-4½	1-5⅞	1-7¼	1-8⅝	1-10	1-11⅜	2-0¾	2-2⅛	2-3½	2-4⅞	2-6¼	2-7⅝	2-9	2-10⅜	2-11¾	3-1⅛	3-2½	3-5¼	3-8	3-10¾	4-1½	4-4¼	4-7	4-9¾	5-0½	5-3¼	5-6	**11**
12	1-1½	1-3	1-4½	1-6	1-7½	1-9	1-10½	2-0	2-1½	2-3	2-4½	2-6	2-7½	2-9	2-10½	3-0	3-1½	3-3	3-4½	3-6	3-9	4-0	4-3	4-6	4-9	5-0	5-3	5-6	5-9	6-0	**12**
13	1-2⅝	1-4¼	1-5⅞	1-7½	1-9⅛	1-10¾	2-0⅜	2-2	2-3⅝	2-5¼	2-6⅞	2-8½	2-10⅛	2-11¾	3-1⅜	3-3	3-4⅝	3-6¼	3-7⅞	3-9½	4-0¾	4-4	4-7¼	4-10½	5-1¾	5-5	5-8¼	5-11½	6-2¾	6-6	**13**
14	1-3¾	1-5½	1-7¼	1-9	1-10¾	2-0½	2-2¼	2-4	2-5¾	2-7½	2-9¼	2-11	3-0¾	3-2½	3-4¼	3-6	3-7¾	3-9½	3-11¼	4-1	4-4½	4-8	4-11½	5-3	5-6½	5-10	6-1½	6-5	6-8½	7-0	**14**
15	1-4⅞	1-6¾	1-8⅝	1-10½	2-0⅜	2-2¼	2-4⅛	2-6	2-7⅞	2-9¾	2-11⅝	3-1½	3-3⅜	3-5¼	3-7⅛	3-9	3-10⅞	4-0¾	4-2⅝	4-4½	4-8¼	5-0	5-3¾	5-7½	5-11¼	6-3	6-6¾	6-10½	7-2¼	7-6	**15**
16	1-6	1-8	1-10	2-0	2-2	2-4	2-6	2-8	2-10	3-0	3-2	3-4	3-6	3-8	3-10	4-0	4-2	4-4	4-6	4-8	5-0	5-4	5-8	6-0	6-4	6-8	7-0	7-4	7-8	8-0	**16**
17	1-7⅛	1-9¼	1-11⅜	2-1½	2-3⅝	2-5¾	2-7⅞	2-10	3-0⅛	3-2¼	3-4⅜	3-6½	3-8⅝	3-10¾	4-0⅞	4-3	4-5⅛	4-7¼	4-9⅜	4-11½	5-3¾	5-8	6-0¼	6-4½	6-8¾	7-1	7-5¼	7-9½	8-1¾	8-6	**17**
18	1-8¼	1-10½	2-0¾	2-3	2-5¼	2-7½	2-9¾	3-0	3-2¼	3-4½	3-6¾	3-9	3-11¼	4-1½	4-3¾	4-6	4-8¼	4-10½	5-0¾	5-3	5-7½	6-0	6-4½	6-9	7-1½	7-6	7-10½	8-3	8-7½	9-0	**18**
19	1-9⅜	1-11¾	2-2⅛	2-4½	2-6⅞	2-9¼	2-11⅝	3-2	3-4⅜	3-6¾	3-9⅛	3-11½	4-1⅞	4-4¼	4-6⅝	4-9	4-11⅜	5-1¾	5-4⅛	5-6½	5-11¼	6-4	6-8¾	7-1½	7-6¼	7-11	8-3¾	8-8½	9-1¼	9-6	**19**
20	1-10½	2-1	2-3½	2-6	2-8½	2-11	3-1½	3-4	3-6½	3-9	3-11½	4-2	4-4½	4-7	4-9½	5-0	5-2½	5-5	5-7½	5-10	6-3	6-8	7-1	7-6	7-11	8-4	8-9	9-2	9-7	10-0	**20**
21	1-11⅝	2-2¼	2-4⅞	2-7½	2-10⅛	3-0¾	3-3⅜	3-6	3-8⅝	3-11¼	4-1⅞	4-4½	4-7⅛	4-9¾	5-0⅜	5-3	5-5⅝	5-8¼	5-10⅞	6-1½	6-6¾	7-0	7-5¼	7-10½	8-3¾	8-9	9-2¼	9-7½	10-0¾	10-6	**21**
22	2-0¾	2-3½	2-6¼	2-9	2-11¾	3-2½	3-5¼	3-8	3-10¾	4-1½	4-4¼	4-7	4-9¾	5-0½	5-3¼	5-6	5-8¾	5-11½	6-2¼	6-5	6-10½	7-4	7-9½	8-3	8-8½	9-2	9-7½	10-1	10-6½	11-0	**22**
23	2-1⅞	2-4¾	2-7⅝	2-10½	3-1⅜	3-4¼	3-7⅛	3-10	4-0⅞	4-3¾	4-6⅝	4-9½	5-0⅜	5-3¼	5-6⅛	5-9	5-11⅞	6-2¾	6-5⅝	6-8½	7-2¼	7-8	8-1¾	8-7½	9-1¼	9-7	10-0¾	10-6½	11-0¼	11-6	**23**
24	2-3	2-6	2-9	3-0	3-3	3-6	3-9	4-0	4-3	4-6	4-9	5-0	5-3	5-6	5-9	6-0	6-3	6-6	6-9	7-0	7-6	8-0	8-6	9-0	9-6	10-0	10-6	11-0	11-6	12-0	**24**
25	2-4⅛	2-7¼	2-10⅜	3-1½	3-4⅝	3-7¾	3-10⅞	4-2	4-5⅛	4-8¼	4-11⅜	5-2½	5-5⅝	5-8¾	5-11⅞	6-3	6-6⅛	6-9¼	7-0⅜	7-3½	7-9¾	8-4	8-10¼	9-4½	9-10¾	10-5	10-11¼	11-5½	11-11¾	12-6	**25**
26	2-5¼	2-8½	2-11¾	3-3	3-6¼	3-9½	4-0¾	4-4	4-7¼	4-10½	5-1¾	5-5	5-8¼	5-11½	6-2¾	6-6	6-9¼	7-0½	7-3¾	7-7	8-1½	8-8	9-2½	9-9	10-3½	10-10	11-4½	11-11	12-5½	13-0	**26**
27	2-6⅜	2-9¾	3-1⅛	3-4½	3-7⅞	3-11¼	4-2⅝	4-6	4-9⅜	5-0¾	5-4⅛	5-7½	5-10⅞	6-2¼	6-5⅝	6-9	7-0⅜	7-3¾	7-7⅛	7-10½	8-5¼	9-0	9-6¾	10-1½	10-8¼	11-3	11-9¾	12-4½	12-11¼	13-6	**27**
28	2-7½	2-11	3-2½	3-6	3-9½	4-1	4-4½	4-8	4-11½	5-3	5-6½	5-10	6-1½	6-5	6-8½	7-0	7-3½	7-7	7-10½	8-2	8-9	9-4	9-11	10-6	11-1	11-8	12-3	12-10	13-5	14-0	**28**
29	2-8⅝	3-0¼	3-3⅞	3-7½	3-11⅛	4-2¾	4-6⅜	4-10	5-1⅝	5-5¼	5-8⅞	6-0½	6-4⅛	6-7¾	6-11⅜	7-3	7-6⅝	7-10¼	8-1⅞	8-5½	9-0¾	9-8	10-3¼	10-10½	11-5¾	12-1	12-8¼	13-3½	13-10¾	14-6	**29**
30	2-9¾	3-1½	3-5¼	3-9	4-0¾	4-4½	4-8¼	5-0	5-3¾	5-7½	5-11¼	6-3	6-6¾	6-10½	7-2¼	7-6	7-9¾	8-1½	8-5¼	8-9	9-4½	10-0	10-7½	11-3	11-10½	12-6	13-1½	13-9	14-4½	15-0	**30**
	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½	2⅝	2¾	2⅞	3	3⅛	3¼	3⅜	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6	

$\frac{5}{8}$" RIVET VALUES

SHEARING AND BEARING VALUES FOR $\frac{5}{8}$" RIVETS IN THOUSANDS OF POUNDS

BEARING VALUES TO THE LEFT OF THE DOTTED LINES ARE LESS THAN THE SINGLE SHEAR VALUES
BEARING VALUES IN PLATES THICKER THAN THOSE GIVEN ARE GREATER THAN THE DOUBLE SHEAR VALUES

$\frac{5}{8}$" RIVETS

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 12000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 24000 LBS./SQ. IN. 3/16	1/4	5/16	3/8	7/16
1	3.7	7.4	2.8	3.8	4.7	5.6	6.6
2	7.4	14.7	5.6	7.5	9.4	11.3	13.1
3	11.0	22.1	8.4	11.3	14.1	16.9	19.7
4	14.7	29.5	11.3	15.0	18.8	22.5	26.2
5	18.4	36.8	14.1	18.8	23.4	28.1	32.8
6	22.1	44.2	16.9	22.5	28.1	33.8	39.4
7	25.8	51.5	19.7	26.3	32.8	39.4	45.9
8	29.5	58.9	22.5	30.0	37.5	45.0	52.5
9	33.1	66.3	25.3	33.8	42.2	50.6	59.1
10	36.8	73.6	28.1	37.5	46.9	56.3	65.6

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 10000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 20000 LBS./SQ. IN. 3/16	1/4	5/16	3/8	7/16
1	3.1	6.1	2.3	3.1	3.9	4.7	5.5
2	6.1	12.3	4.7	6.3	7.8	9.4	10.9
3	9.2	18.4	7.0	9.4	11.7	14.1	16.4
4	12.3	24.5	9.4	12.5	15.6	18.8	21.9
5	15.3	30.7	11.7	15.6	19.5	23.4	27.3
6	18.4	36.8	14.1	18.8	23.4	28.1	32.8
7	21.5	43.0	16.4	21.9	27.3	32.8	38.3
8	24.5	49.1	18.8	25.0	31.3	37.5	43.8
9	27.6	55.2	21.1	28.1	35.2	42.2	49.2
10	30.7	61.4	23.4	31.3	39.1	46.9	54.7

BOLTS

NO. OF RIVS.	SHEAR AT 9000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 18000 LBS./SQ. IN. 3/16	1/4	5/16	3/8	7/16
1	2.8	5.5	2.1	2.8	3.5	4.2	4.9
2	5.5	11.0	4.2	5.6	7.0	8.4	9.8
3	8.3	16.6	6.3	8.4	10.5	12.7	14.8
4	11.0	22.1	8.4	11.2	14.1	16.9	19.7
5	13.8	27.6	10.6	14.1	17.6	21.1	24.6
6	16.6	33.1	12.7	16.9	21.1	25.3	29.5
7	19.3	38.7	14.8	19.7	24.6	29.5	34.5
8	22.1	44.2	16.9	22.5	28.1	33.8	39.4
9	24.8	49.7	19.0	25.3	31.6	38.0	44.3
10	27.6	55.2	21.1	28.1	35.2	42.2	49.2

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 11000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 22000 LBS./SQ. IN. 3/16	1/4	5/16	3/8	7/16
1	3.4	6.8	2.6	3.4	4.3	5.2	6.0
2	6.8	13.5	5.2	6.9	8.6	10.3	12.0
3	10.1	20.3	7.7	10.3	12.9	15.5	18.0
4	13.5	27.0	10.3	13.7	17.2	20.6	24.1
5	16.9	33.8	12.9	17.2	21.5	25.8	30.1
6	20.3	40.5	15.5	20.6	25.8	30.9	36.1
7	23.6	47.3	18.0	24.1	30.1	36.1	42.1
8	27.0	54.0	20.6	27.5	34.4	41.2	48.1
9	30.4	60.8	23.2	30.9	38.7	46.4	54.1
10	33.8	67.5	25.8	34.4	43.0	51.6	60.2

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 9000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 18000 LBS./SQ. IN. 3/16	1/4	5/16	3/8	7/16
1	2.8	5.5	2.1	2.8	3.5	4.2	4.9
2	5.5	11.0	4.2	5.6	7.0	8.4	9.8
3	8.3	16.6	6.3	8.4	10.5	12.7	14.8
4	11.0	22.1	8.4	11.2	14.1	16.9	19.7
5	13.8	27.6	10.6	14.1	17.6	21.1	24.6
6	16.6	33.1	12.7	16.9	21.1	25.3	29.5
7	19.3	38.7	14.8	19.7	24.6	29.5	34.5
8	22.1	44.2	16.9	22.5	28.1	33.8	39.4
9	24.8	49.7	19.0	25.3	31.6	38.0	44.3
10	27.6	55.2	21.1	28.1	35.2	42.2	49.2

BOLTS

NO. OF RIVS.	SHEAR AT 8000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 16000 LBS./SQ. IN. 3/16	1/4	5/16	3/8	7/16
1	2.5	4.9	1.9	2.5	3.1	3.8	4.4
2	4.9	9.8	3.8	5.0	6.3	7.5	8.8
3	7.4	14.7	5.6	7.5	9.4	11.3	13.1
4	9.8	19.6	7.5	10.0	12.5	15.0	17.5
5	12.3	24.5	9.4	12.5	15.6	18.8	21.9
6	14.7	29.5	11.3	15.0	18.8	22.5	26.3
7	17.2	34.4	13.1	17.5	21.9	26.3	30.
8	19.6	39.3	15.0	20.0	25.0	30.0	35.
9	22.1	44.2	16.9	22.5	28.1	33.8	39
10	24.5	49.1	18.8	25.0	31.3	37.5	4

SHOP RIVETS

NO. OF RIVS.	SHEAR AT [illegible] /SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 20000 LBS./SQ. IN. 3/16	1/4	5/16	3/8	7/16
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.7	5.5
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	10.9

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 8000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 16000 LBS./SQ. IN. 3/16	1/4	5/16	3/8	7/16
1	2.5	4.9	1.9	2.5	3.1	3.8	4.4
2	4.9	9.8	3.8	5.0	6.3	7.5	8.8
3	7.4	14.7	5.6	7.5	9.4	11.3	13.1
4	9.8	19.6	7.5	10.0	12.5	15.0	17.5
5	[illegible]	24.5	9.4	12.5	15.6	18.8	21.9
6	[illegible]	[illegible]	11.3	15.0	18.8	22.5	26.3
7	[illegible]	[illegible]	[illegible]	17.5	21.9	26.3	30.6
8	[illegible]	[illegible]	[illegible]	[illegible]	25.0	30.0	35.0
9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	33.8	39.4
10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	43.8

BOLTS

NO. OF RIVS.	SHEAR AT 7000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 14000 LBS./ 3/16	1/4	5/16	3/8
1	2.2	4.3	1.6	2.2	2.7	3.3
2	4.3	8.6	3.3	4.4	5.5	6.6
3	6.4	12.9	4.9	6.6	8.2	9.8
4	8.6	17.2	6.6	8.8	10.9	13.1
5	10.7	21.5	8.2	10.9	13.7	16.4
6	12.9	25.8	9.8	13.1	16.4	19
7	15.0	30.1	11.5	15.3	19.1	23
8	17.2	34.4	13.1	17.5	21.9	26
9	19.3	38.7	14.8	19.7	24.6	2
10	21.5	43.0	16.4	21.9	27.3	[illegible]

SHEARING AND BEARING VALUES FOR $\frac{3}{4}''$ RIVETS IN THOUSANDS OF POUNDS

$\frac{3}{4}''$ RIVETS | $\frac{3}{4}''$ RIVETS

BEARING VALUES TO THE LEFT OF THE DOTTED LINES ARE LESS THAN THE SINGLE SHEAR VALUES
BEARING VALUES IN PLATES THICKER THAN THOSE GIVEN ARE GREATER THAN THE DOUBLE SHEAR VALUES

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 12000 LBS./SQ. IN.		BEARING IN PLATE AT 24000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	5.3	10.6	3.4	4.5	5.6	6.8	7.9	9.0	10.1
2	10.6	21.2	6.8	9.0	11.3	13.5	15.8	18.0	20.3
3	15.9	31.8	10.1	13.5	16.9	20.3	23.6	27.0	30.4
4	21.2	42.4	13.5	18.0	22.5	27.0	31.5	36.0	40.5
5	26.5	53.0	16.9	22.5	28.1	33.8	39.4	45.0	50.6
6	31.8	63.6	20.3	27.0	33.8	40.5	47.3	54.0	60.8
7	37.1	74.2	23.6	31.5	39.4	47.3	55.1	63.0	70.9
8	42.4	84.8	27.0	36.0	45.0	54.0	63.0	72.0	81.0
9	47.7	95.4	30.4	40.5	50.6	60.8	70.9	81.0	91.1
10	53.0	106.0	33.8	45.0	56.3	67.5	78.8	90.0	101.3

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 10000 LBS./SQ. IN.		BEARING IN PLATE AT 20000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	4.4	8.8	2.8	3.8	4.7	5.6	6.6	7.5	8.4
2	8.8	17.7	5.6	7.5	9.4	11.3	13.1	15.0	16.9
3	13.3	26.5	8.4	11.3	14.1	16.9	19.7	22.5	25.3
4	17.7	35.3	11.3	15.0	18.8	22.5	26.3	30.0	33.8
5	22.1	44.2	14.1	18.8	23.4	28.1	32.8	37.5	42.2
6	26.5	53.0	16.9	22.5	28.1	33.8	39.4	45.0	50.6
7	30.9	61.9	19.7	26.3	32.8	39.4	45.9	52.5	59.1
8	35.3	70.7	22.5	30.0	37.5	45.0	52.5	60.0	67.5
9	39.8	79.5	25.3	33.8	42.2	50.6	59.1	67.5	75.9
10	44.2	88.4	28.1	37.5	46.9	56.3	65.6	75.0	84.4

BOLTS

NO. OF RIVS.	SHEAR AT 9000 LBS./SQ. IN.		BEARING IN PLATE AT 18000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	4.0	8.0	2.5	3.4	4.2	5.1	5.9	6.8	7.6
2	8.0	15.9	5.1	6.8	8.4	10.1	11.8	13.5	15.2
3	11.9	23.9	7.6	10.1	12.7	15.2	17.7	20.3	22.8
4	15.9	31.8	10.1	13.5	16.9	20.3	23.6	27.0	30.4
5	19.9	39.8	12.7	16.9	21.1	25.3	29.5	33.8	38.0
6	23.9	47.7	15.2	20.3	25.3	30.4	35.4	40.5	45.6
7	27.8	55.7	17.7	23.6	29.5	35.4	41.3	47.3	53.2
8	31.8	63.6	20.2	27.0	33.8	40.5	47.2	54.0	60.8
9	35.8	71.6	22.8	30.4	38.0	45.6	53.2	60.8	68.3
10	39.8	79.5	25.3	33.8	42.2	50.6	59.1	67.5	75.9

SHOP RIVETS

NO. OF RIVS	SHEAR AT 11000 LBS./SQ. IN.		BEARING IN PLATE AT 22000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	4.9	9.7	3.1	4.1	5.2	6.2	7.2	8.3	9.3
2	9.7	19.4	6.2	8.3	10.3	12.4	14.4	16.5	18.6
3	14.6	29.2	9.3	12.4	15.5	18.6	21.7	24.8	27.8
4	19.4	38.9	12.4	16.5	20.6	24.8	28.9	33.0	37.1
5	24.3	48.6	15.5	20.6	25.8	30.9	36.1	41.3	46.4
6	29.2	58.3	18.6	24.8	30.9	37.1	43.3	49.5	55.7
7	34.0	68.0	21.7	28.9	36.1	43.3	50.5	57.8	65.0
8	38.9	77.8	24.8	33.0	41.2	49.5	57.8	66.0	74.2
9	43.7	87.5	27.8	37.1	46.4	55.7	65.0	74.3	83.5
10	48.6	97.2	30.9	41.3	51.6	61.9	72.2	82.5	92.8

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 9000 LBS./SQ. IN.		BEARING IN PLATE AT 18000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	4.0	8.0	2.5	3.4	4.2	5.1	5.9	6.8	7.6
2	8.0	15.9	5.1	6.8	8.4	10.1	11.8	13.5	15.2
3	11.9	23.9	7.6	10.1	12.7	15.2	17.7	20.3	22.8
4	15.9	31.8	10.1	13.5	16.9	20.3	23.6	27.0	30.4
5	19.9	39.8	12.7	16.9	21.1	25.3	29.5	33.8	38.0
6	23.9	47.7	15.2	20.3	25.3	30.4	35.4	40.5	45.6
7	27.8	55.7	17.7	23.6	29.5	35.4	41.3	47.3	53.2
8	31.8	63.6	20.2	27.0	33.8	40.5	47.2	54.0	60.8
9	35.8	71.6	22.8	30.4	38.0	45.6	53.2	60.8	68.3
10	39.8	79.5	25.3	33.8	42.2	50.6	59.1	67.5	75.9

BOLTS

NO. OF RIVS.	SHEAR AT 8000 LBS./SQ. IN.		BEARING IN PLATE AT 16000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	3.5	7.1	2.3	3.0	3.8	4.5	5.3	6.0	6.8
2	7.1	14.1	4.5	6.0	7.5	9.0	10.5	12.0	13.5
3	10.6	21.2	6.8	9.0	11.3	13.5	15.8	18.0	20.3
4	14.1	28.3	9.0	12.0	15.0	18.0	21.0	24.0	27.0
5	17.7	35.3	11.3	15.0	18.8	22.5	26.3	30.0	33.8
6	21.2	42.4	13.5	18.0	22.5	27.0	31.5	36.0	40.5
7	24.7	49.5	15.8	21.0	26.3	31.5	36.8	42.0	47.3
8	28.3	56.6	18.0	24.0	30.0	36.0	42.0	48.0	54.0
9	31.8	63.6	20.3	27.0	33.8	40.5	47.3	54.0	60.8
10	35.3	70.7	22.5	30.0	37.5	45.0	52.5	60.0	67.5

SHOP RIVETS

NO. OF RIVS	SHEAR AT 10000 LBS./SQ. IN.		BEARING IN PLATE AT 20000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	4.4	8.8	2.8	3.8	4.7	5.6	6.6	7.5	8.4
2	8.8	17.7	5.6	7.5	9.4	11.3	13.1	15.0	16.9
3	13.3	26.5	8.4	11.3	14.1	16.9	19.7	22.5	25.3
4	17.7	35.3	11.3	15.0	18.8	22.5	26.3	30.0	33.8
5	22.1	44.2	14.1	18.8	23.4	28.1	32.1	37.5	42.2
6	26.5	53.0	16.9	22.5	28.1	33.8	39.4	45.0	50.6
7	30.9	61.9	19.7	26.3	32.8	39.4	45.9	52.5	59.1
8	35.3	70.7	22.5	30.0	37.5	45.0	52.5	60.0	67.5
9	39.8	79.5	25.3	33.8	42.2	50.6	59.1	67.5	75.9
10	44.2	88.4	28.1	37.5	46.9	56.3	65.6	75.0	84.4

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 8000 LBS./SQ. IN.		BEARING IN PLATE AT 16000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	3.5	7.1	2.3	3.0	3.8	4.5	5.3	6.0	6.8
2	7.1	14.1	4.5	6.0	7.5	9.0	10.5	12.0	13.5
3	10.6	21.2	6.8	9.0	11.3	13.5	15.8	18.0	20.3
4	14.1	28.3	9.0	12.0	15.0	18.0	21.0	24.0	27.0
5	17.7	35.3	11.3	15.0	18.8	22.5	26.3	30.0	33.8
6	21.2	42.4	13.5	18.0	22.5	27.0	31.5	36.0	40.5
7	24.7	49.5	15.8	21.0	26.3	31.5	36.8	42.0	47.3
8	28.3	56.6	18.0	24.0	30.0	36.0	42.0	48.0	54.0
9	31.8	63.6	20.3	27.0	33.8	40.5	47.3	54.0	60.8
10	35.3	70.7	22.5	30.0	37.5	45.0	52.5	60.0	67.5

BOLTS

NO. OF RIVS.	SHEAR AT 7000 LBS./SQ. IN.		BEARING IN PLATE AT 14000 LBS./SQ. IN.						
	SINGLE	DOUBLE	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
1	3.1	6.2	2.0	2.6	3.3	3.9	4.6	5.3	5.9
2	6.2	12.4	3.9	5.3	6.6	7.9	9.2	10.5	11.8
3	9.3	18.6	5.9	7.9	9.8	11.8	13.8	15.8	17.7
4	12.4	24.7	7.9	10.5	13.1	15.8	18.4	21.0	23.6
5	15.5	30.9	9.8	13.1	16.4	19.7	23.0	26.3	29.5
6	18.6	37.1	11.8	15.8	19.7	23.6	27.6	31.5	35.4
7	21.7	43.3	13.8	18.4	23.0	27.6	32.2	36.8	41.3
8	24.7	49.5	15.8	21.0	26.3	31.5	36.8	42.0	47.2
9	27.8	56.7	17.7	23.6	29.5	35.4	41.3	47.3	53.2
10	30.9	61.9	19.7	26.3	32.8	39.4	45.9	52.5	59.1

$\frac{7}{8}''$ RIVETS — SHEARING AND BEARING VALUES FOR $\frac{7}{8}''$ RIVETS IN THOUSANDS OF POUNDS — $\frac{7}{8}''$ RIVETS

BEARING VALUES TO THE LEFT OF THE DOTTED LINES ARE LESS THAN THE SINGLE SHEAR VALUES
BEARING VALUES IN PLATES THICKER THAN THOSE GIVEN ARE GREATER THAN THE DOUBLE SHEAR VALUES

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 12000 LBS./SQ. IN.		BEARING IN PLATE AT 24000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	7.2	14.4	5.3	6.6	7.9	9.2	10.5	11.8	13.1
2	14.4	28.9	10.5	13.1	15.8	18.4	21.0	23.6	26.3
3	21.6	43.3	15.8	19.7	23.6	27.6	31.5	35.4	39.4
4	28.9	57.7	21.0	26.3	31.5	36.8	42.0	47.2	52.5
5	36.1	72.2	26.3	32.8	39.4	45.9	52.5	59.1	65.6
6	43.3	86.6	31.5	39.4	47.3	55.1	63.0	70.9	78.8
7	50.5	101.0	36.8	45.9	55.1	64.3	73.5	82.7	91.9
8	57.7	115.4	42.0	52.5	63.0	73.5	84.0	94.5	105.0
9	64.9	129.9	47.3	59.1	70.9	82.7	94.5	106.3	118.1
10	72.2	144.3	52.5	65.6	78.8	91.9	105.0	118.1	131.3

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 10000 LBS./SQ. IN.		BEARING IN PLATE AT 20000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	6.0	12.0	4.4	5.5	6.6	7.7	8.8	9.8	10.9
2	12.0	24.1	8.8	10.9	13.1	15.3	17.5	19.7	21.9
3	18.0	36.1	13.1	16.4	19.7	23.0	26.3	29.5	32.8
4	24.1	48.1	17.5	21.9	26.3	30.6	35.0	39.4	43.8
5	30.1	60.1	21.9	27.3	32.8	38.3	43.8	49.2	54.7
6	36.1	72.2	26.3	32.8	39.4	45.9	52.5	59.1	65.6
7	42.1	84.2	30.6	38.3	45.9	53.6	61.3	68.9	76.6
8	48.1	96.2	35.0	43.8	52.5	61.2	70.0	78.8	87.5
9	54.1	108.2	39.4	49.2	59.1	68.9	78.8	88.6	98.4
10	60.1	120.3	43.8	54.7	65.6	76.6	87.5	98.4	109.4

BOLTS

NO. OF RIVS.	SHEAR AT 9000 LBS./SQ. IN.		BEARING IN PLATE AT 18000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	5.4	10.8	3.9	4.9	5.9	6.9	7.9	8.9	9.8
2	10.8	21.6	7.9	9.8	11.8	13.8	15.8	17.7	19.7
3	16.2	32.5	11.8	14.8	17.7	20.7	23.6	26.6	29.5
4	21.6	43.3	15.8	19.7	23.6	27.6	31.5	35.4	39.4
5	27.1	54.1	19.7	24.6	29.5	34.5	39.4	44.3	49.2
6	32.5	64.9	23.6	29.5	35.4	41.3	47.3	53.2	59.1
7	37.9	75.8	27.6	34.5	41.3	48.2	55.1	62.0	68.9
8	43.3	86.6	31.5	39.4	47.2	55.1	63.0	70.9	78.8
9	48.7	97.4	35.4	44.3	53.2	62.0	70.9	79.7	88.6
10	54.1	108.2	39.4	49.2	59.1	68.9	78.8	88.6	98.4

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 11000 LBS./SQ. IN.		BEARING IN PLATE AT 22000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	6.6	13.2	4.8	6.0	7.2	8.4	9.6	10.8	12.0
2	13.2	26.5	9.6	12.0	14.4	16.8	19.3	21.7	24.1
3	19.8	39.7	14.4	18.0	21.7	25.3	28.9	32.5	36.1
4	26.5	52.9	19.2	24.1	28.9	33.7	38.5	43.3	48.1
5	33.1	66.1	24.1	30.1	36.1	42.1	48.1	54.1	60.2
6	39.7	79.4	28.9	36.1	43.3	50.5	57.8	65.0	72.2
7	46.3	92.6	33.7	42.1	50.5	59.0	67.4	75.8	84.2
8	52.9	105.8	38.5	48.1	57.8	67.4	77.0	86.6	96.2
9	59.5	119.1	43.3	54.1	65.0	75.8	86.6	97.5	108.3
10	66.1	132.3	48.1	60.2	72.2	84.2	96.3	108.3	120.3

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 9000 LBS./SQ. IN.		BEARING IN PLATE AT 18000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	5.4	10.8	3.9	4.9	5.9	6.9	7.9	8.9	9.8
2	10.8	21.6	7.9	9.8	11.8	13.8	15.8	17.7	19.7
3	16.2	32.5	11.8	14.8	17.7	20.7	23.6	26.6	29.5
4	21.6	43.3	15.8	19.7	23.6	27.6	31.5	35.4	39.4
5	27.1	54.1	19.7	24.6	29.5	34.5	39.4	44.3	49.2
6	32.5	64.9	23.6	29.5	35.4	41.3	47.3	53.2	59.1
7	37.9	75.8	27.6	34.5	41.3	48.2	55.1	62.0	68.9
8	43.3	86.6	31.5	39.4	47.2	55.1	63.0	70.9	78.8
9	48.7	97.4	35.4	44.3	53.2	62.0	70.9	79.7	88.6
10	54.1	108.2	39.4	49.2	59.1	68.9	78.8	88.6	98.4

BOLTS

NO. OF RIVS.	SHEAR AT 8000 LBS./SQ. IN.		BEARING IN PLATE AT 16000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	4.8	9.6	3.5	4.4	5.3	6.1	7.0	7.9	8.8
2	9.6	19.2	7.0	8.8	10.5	12.3	14.0	15.8	17.5
3	14.4	28.9	10.5	13.1	15.8	18.4	21.0	23.6	26.3
4	19.2	38.5	14.0	17.5	21.0	24.5	28.0	31.5	35.0
5	24.1	48.1	17.5	21.9	26.3	30.6	35.0	39.4	43.8
6	28.9	57.7	21.0	26.3	31.5	36.8	42.0	47.3	52.5
7	33.7	67.3	24.5	30.6	36.8	42.9	49.0	55.1	61.3
8	38.5	77.0	28.0	35.0	42.0	49.0	56.0	63.0	70.0
9	43.3	86.6	31.5	39.4	47.3	55.1	63.0	70.9	78.8
10	48.1	96.2	35.0	43.8	52.5	61.3	70.0	78.8	87.5

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 10000 LBS./SQ. IN.		BEARING IN PLATE AT 20000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	6.0	12.0	4.4	5.5	6.6	7.7	8.8	9.8	10.9
2	12.0	24.1	8.8	10.9	13.1	15.3	17.5	19.7	21.9
3	18.0	36.1	13.1	16.4	19.7	23.0	26.3	29.5	32.8
4	24.1	48.1	17.5	21.9	26.3	30.6	35.0	39.4	43.8
5	30.1	60.1	21.9	27.3	32.8	38.3	43.8	49.2	54.7
6	36.1	72.2	26.3	32.8	39.4	45.9	52.5	59.1	65.6
7	42.1	84.2	30.6	38.3	45.9	53.6	61.3	68.9	76.6
8	48.1	96.2	35.0	43.8	52.5	61.2	70.0	78.8	87.5
9	54.1	108.2	39.4	49.2	59.1	68.9	78.8	88.6	98.4
10	60.1	120.3	43.8	54.7	65.6	76.6	87.5	98.4	109.4

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 8000 LBS./SQ. IN.		BEARING IN PLATE AT 16000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	4.8	9.6	3.5	4.4	5.3	6.1	7.0	7.9	8.8
2	9.6	19.2	7.0	8.8	10.5	12.3	14.0	15.8	17.5
3	14.4	28.9	10.5	13.1	15.8	18.4	21.0	23.6	26.3
4	19.2	38.5	14.0	17.5	21.0	24.5	28.0	31.5	35.0
5	24.1	48.1	17.5	21.9	26.3	30.6	35.0	39.4	43.8
6	28.9	57.7	21.0	26.3	31.5	36.8	42.0	47.3	52.5
7	33.7	67.3	24.5	30.6	36.8	42.9	49.0	55.1	61.3
8	38.5	77.0	28.0	35.0	42.0	49.0	56.0	63.0	70.0
9	43.3	86.6	31.5	39.4	47.3	55.1	63.0	70.9	78.8
10	48.1	96.2	35.0	43.8	52.5	61.3	70.0	78.8	87.5

BOLTS

NO. OF RIVS.	SHEAR AT 7000 LBS./SQ. IN.		BEARING IN PLATE AT 14000 LBS./SQ. IN.						
	SINGLE	DOUBLE	1/4	5/16	3/8	7/16	1/2	9/16	5/8
1	4.2	8.4	3.1	3.8	4.6	5.4	6.1	6.9	7.7
2	8.4	16.8	6.1	7.7	9.2	10.7	12.3	13.8	15.3
3	12.6	25.3	9.2	11.5	13.8	16.1	18.4	20.7	23.0
4	16.8	33.7	12.3	15.3	18.4	21.4	24.5	27.6	30.6
5	21.0	42.1	15.3	19.1	23.0	26.8	30.6	34.5	38.3
6	25.3	50.5	18.4	23.0	27.6	32.2	36.8	41.3	45.9
7	29.5	58.9	21.4	26.8	32.2	37.5	42.9	48.2	53.6
8	33.7	67.3	24.5	30.6	36.8	42.9	49.0	55.1	61.2
9	37.9	75.8	27.6	34.5	41.3	48.2	55.1	62.0	68.9
10	42.1	84.2	30.6	38.3	45.9	53.6	61.3	68.9	76.6

SHEARING AND BEARING VALUES FOR 1″ RIVETS IN THOUSANDS OF POUNDS

1″ RIVETS — 1″ RIVETS

BEARING VALUES TO THE LEFT OF THE DOTTED LINES ARE LESS THAN THE SINGLE SHEAR VALUES
BEARING VALUES IN PLATES THICKER THAN THOSE GIVEN ARE GREATER THAN THE DOUBLE SHEAR VALUES

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 12000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 24000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	9.4	18.9	9.0	10.5	12.0	13.5	15.0	16.5	18.0
2	18.9	37.7	18.0	21.0	24.0	27.0	30.0	33.0	36.0
3	28.3	56.6	27.0	31.5	36.0	40.5	45.0	49.5	54.0
4	37.7	75.4	36.0	42.0	48.0	54.0	60.0	66.0	72.0
5	47.1	94.3	45.0	52.5	60.0	67.5	75.0	82.5	90.0
6	56.6	113.1	54.0	63.0	72.0	81.0	90.0	99.0	108.0
7	66.0	132.0	63.0	73.5	84.0	94.5	105.0	115.5	126.0
8	75.4	150.8	72.0	84.0	96.0	108.0	120.0	132.0	144.0
9	84.8	169.7	81.0	94.5	108.0	121.5	135.0	148.5	162.0
10	94.2	188.5	90.0	105.0	120.0	135.0	150.0	165.0	180.0

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 10000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 20000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	7.9	15.7	7.5	8.8	10.0	11.3	12.5	13.8	15.0
2	15.7	31.4	15.0	17.5	20.0	22.5	25.0	27.5	30.0
3	23.6	47.1	22.5	26.3	30.0	33.8	37.5	41.3	45.0
4	31.4	62.8	30.0	35.0	40.0	45.0	50.0	55.0	60.0
5	39.3	78.5	37.5	43.8	50.0	56.3	62.5	68.8	75.0
6	47.1	94.2	45.0	52.5	60.0	67.5	75.0	82.5	90.0
7	55.0	110.0	52.5	61.3	70.0	78.8	87.5	96.3	105.0
8	62.8	125.7	60.0	70.0	80.0	90.0	100.0	110.0	120.0
9	70.7	141.4	67.5	78.8	90.0	101.3	112.5	123.8	135.0
10	78.5	157.1	75.0	87.5	100.0	112.5	125.0	137.5	150.0

BOLTS

NO. OF RIVS.	SHEAR AT 9000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 18000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	7.1	14.1	6.8	7.9	9.0	10.1	11.3	12.4	13.5
2	14.1	28.3	13.5	15.8	18.0	20.3	22.5	24.8	27.0
3	21.2	42.4	20.3	23.6	27.0	30.4	33.8	37.1	40.5
4	28.3	56.5	27.0	31.5	36.0	40.5	45.0	49.5	54.0
5	35.3	70.7	33.8	39.4	45.0	50.6	56.3	61.9	67.5
6	42.4	84.8	40.5	47.3	54.0	60.8	67.5	74.3	81.0
7	49.5	99.0	47.3	55.1	63.0	70.9	78.8	86.6	94.5
8	56.6	113.1	54.0	63.0	72.0	81.0	90.0	99.0	108.0
9	63.6	127.2	60.8	70.9	81.0	91.1	101.3	111.4	121.5
10	70.7	141.4	67.5	78.8	90.0	101.3	112.5	123.8	135.0

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 11000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 22000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	8.6	17.3	8.3	9.6	11.0	12.4	13.8	15.1	16.5
2	17.3	34.6	16.5	19.3	22.0	24.8	27.5	30.3	33.0
3	25.9	51.8	24.8	28.9	33.0	37.1	41.3	45.4	49.5
4	34.6	69.1	33.0	38.5	44.0	49.5	55.0	60.5	66.0
5	43.2	86.4	41.3	48.1	55.0	61.9	68.8	75.6	82.5
6	51.8	103.7	49.5	57.8	66.0	74.3	82.5	90.8	99.0
7	60.5	121.0	57.8	67.4	77.0	86.6	96.3	105.9	115.5
8	69.1	138.2	66.0	77.0	88.0	99.0	110.0	121.0	132.0
9	77.8	155.5	74.3	86.6	99.0	111.4	123.8	136.1	148.5
10	86.4	172.8	82.5	96.3	110.0	123.8	137.5	151.3	165.0

FIELD RIVETS

NO. OF RIVS.	SHEAR AT 9000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 18000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	7.1	14.1	6.8	7.9	9.0	10.1	11.3	12.4	13.5
2	14.1	28.3	13.5	15.8	18.0	20.3	22.5	24.8	27.0
3	21.2	42.4	20.3	23.6	27.0	30.4	33.8	37.1	40.5
4	28.3	56.5	27.0	31.5	36.0	40.5	45.0	49.5	54.0
5	35.3	70.7	33.8	39.4	45.0	50.6	56.3	61.9	67.5
6	42.4	84.8	40.5	47.3	54.0	60.8	67.5	74.3	81.0
7	49.5	99.0	47.3	55.1	63.0	70.9	78.8	86.6	94.5
8	56.6	113.1	54.0	63.0	72.0	81.0	90.0	99.0	108.0
9	63.6	127.2	60.8	70.9	81.0	91.1	101.3	111.4	121.5
10	70.7	141.4	67.5	78.8	90.0	101.3	112.5	123.8	135.0

BOLTS

NO. OF RIVS.	SHEAR AT 8000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 16000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	6.3	12.6	6.0	7.0	8.0	9.0	10.0	11.0	12.0
2	12.6	25.1	12.0	14.0	16.0	18.0	20.0	22.0	24.0
3	18.8	37.7	18.0	21.0	24.0	27.0	30.0	33.0	36.0
4	25.1	50.3	24.0	28.0	32.0	36.0	40.0	44.0	48.0
5	31.4	62.8	30.0	35.0	40.0	45.0	50.0	55.0	60.0
6	37.7	75.4	36.0	42.0	48.0	54.0	60.0	66.0	72.0
7	44.0	88.0	42.0	49.0	56.0	63.0	70.0	77.0	84.0
8	50.3	100.5	48.0	56.0	64.0	72.0	80.0	88.0	96.0
9	56.5	113.1	54.0	63.0	72.0	81.0	90.0	99.0	108.0
10	62.8	125.7	60.0	70.0	80.0	90.0	100.0	110.0	120.0

SHOP RIVETS

NO. OF RIVS.	SHEAR AT 10000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 20000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	7.9	15.7	7.5	8.8	10.0	11.3	12.5	13.8	15.0
2	15.7	31.4	15.0	17.5	20.0	22.5	25.0	27.5	30.0
3	23.6	47.1	22.5	26.3	30.0	33.8	37.5	41.3	45.0
4	31.4	62.8	30.0	35.0	40.0	45.0	50.0	55.0	60.0
5	39.3	78.5	37.5	43.8	50.0	56.3	62.5	68.8	75.0
6	47.1	94.2	45.0	52.5	60.0	67.5	75.0	82.5	90.0
7	55.0	110.0	52.5	61.3	70.0	78.8	87.5	96.3	105.0
8	62.8	125.7	60.0	70.0	80.0	90.0	100.0	110.0	120.0
9	70.7	141.4	67.5	78.8	90.0	101.3	112.5	123.8	135.0
10	78.5	157.1	75.0	87.5	100.0	112.5	125.0	137.5	150.0

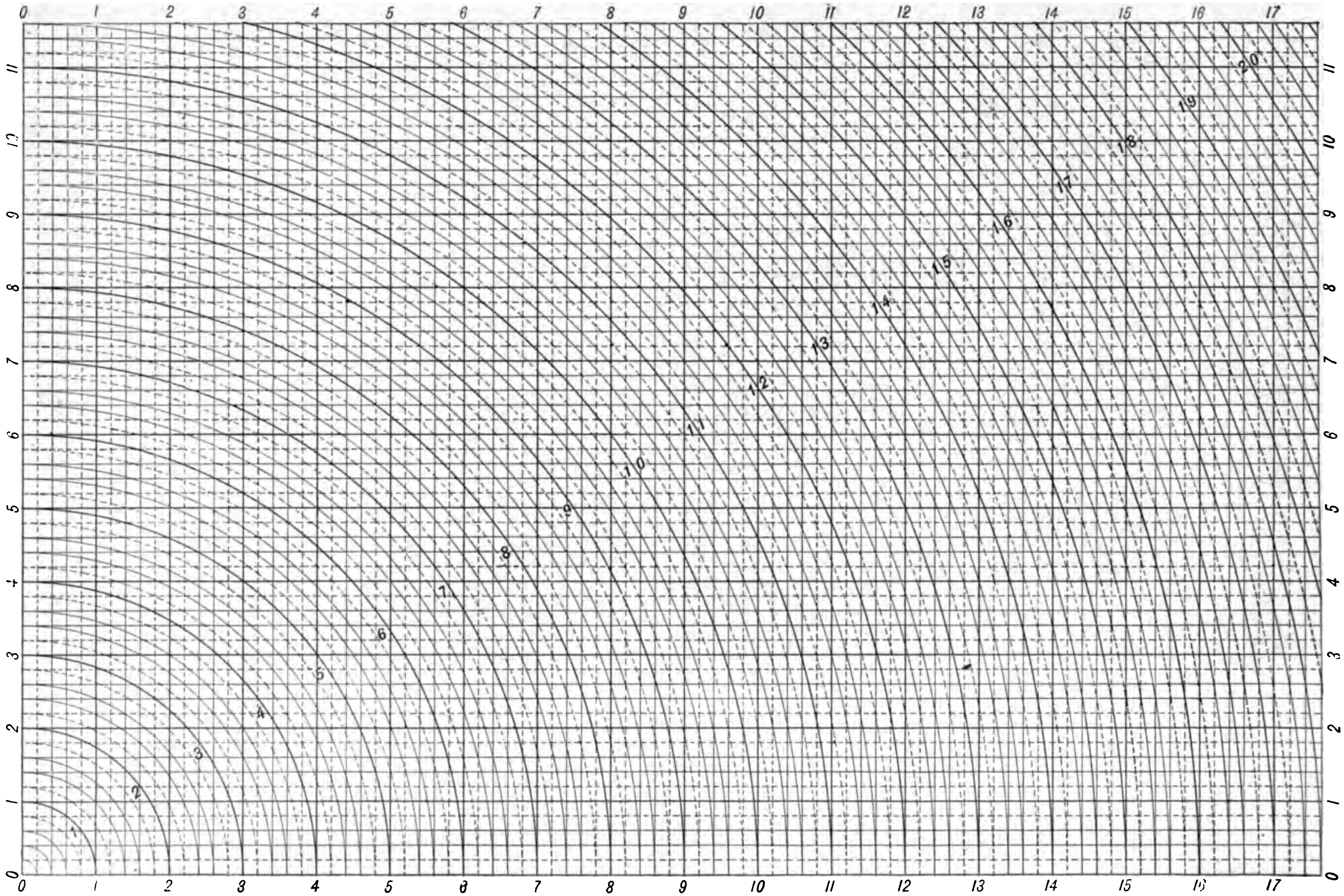
FIELD RIVETS

NO. OF RIVS.	SHEAR AT 8000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 16000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	6.3	12.6	6.0	7.0	8.0	9.0	10.0	11.0	12.0
2	12.6	25.1	12.0	14.0	16.0	18.0	20.0	22.0	24.0
3	18.8	37.7	18.0	21.0	24.0	27.0	30.0	33.0	36.0
4	25.1	50.3	24.0	28.0	32.0	36.0	40.0	44.0	48.0
5	31.4	62.8	30.0	35.0	40.0	45.0	50.0	55.0	60.0
6	37.7	75.4	36.0	42.0	48.0	54.0	60.0	66.0	72.0
7	44.0	88.0	42.0	49.0	56.0	63.0	70.0	77.0	84.0
8	50.3	100.5	48.0	56.0	64.0	72.0	80.0	88.0	96.0
9	56.5	113.1	54.0	63.0	72.0	81.0	90.0	99.0	108.0
10	62.8	125.7	60.0	70.0	80.0	90.0	100.0	110.0	120.0

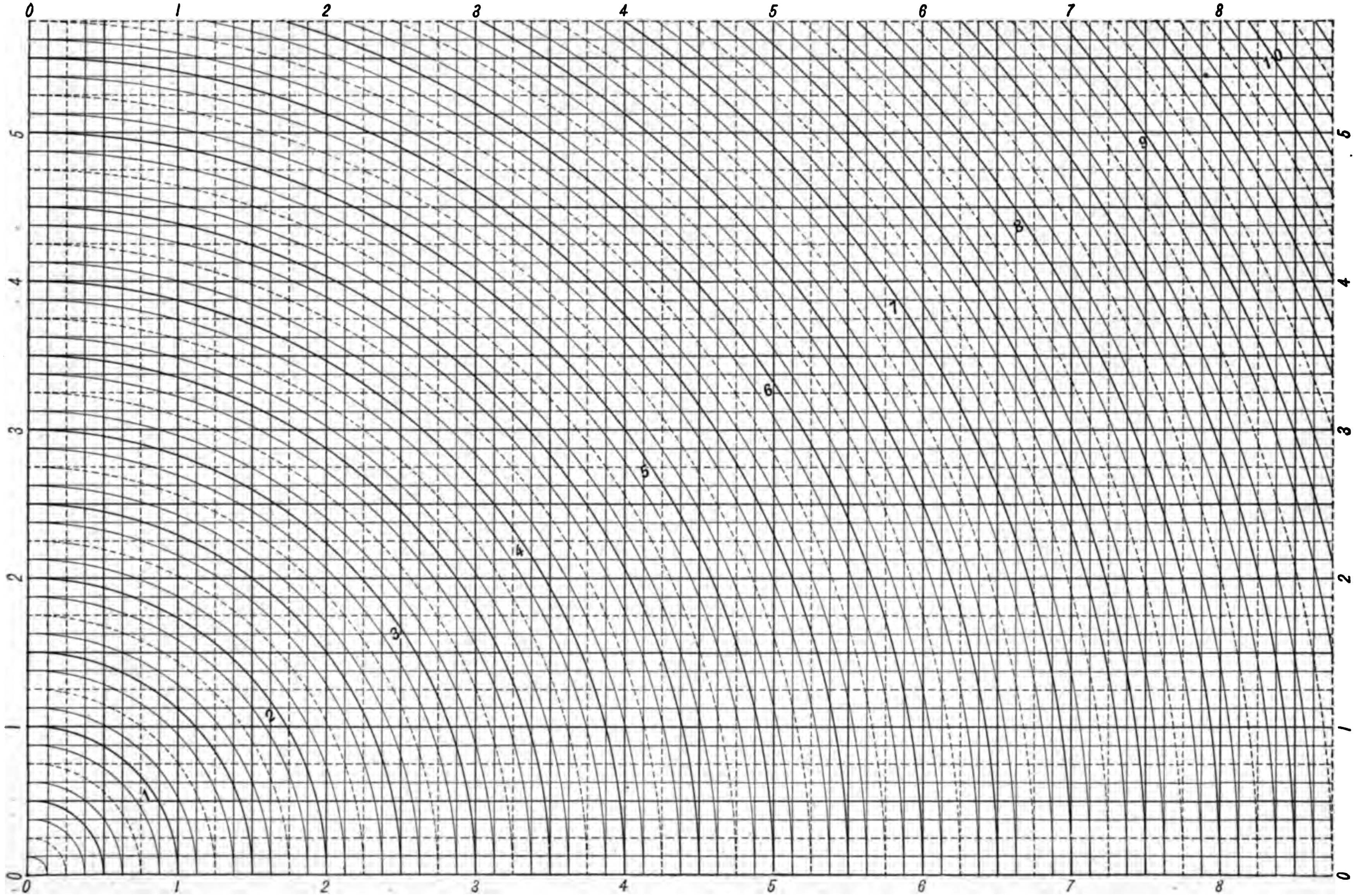
BOLTS

NO. OF RIVS.	SHEAR AT 7000 LBS./SQ. IN. SINGLE	DOUBLE	BEARING IN PLATE AT 14000 LBS./SQ. IN. 3/8	7/16	1/2	9/16	5/8	11/16	3/4
1	5.5	11.0	5.3	6.1	7.0	7.9	8.8	9.6	10.5
2	11.0	22.0	10.5	12.3	14.0	15.8	17.5	19.3	21.0
3	16.5	33.0	15.8	18.4	21.0	23.6	26.3	28.9	31.5
4	22.0	44.0	21.0	24.5	28.0	31.5	35.0	38.5	42.0
5	27.5	55.0	26.3	30.6	35.0	39.4	43.8	48.1	52.5
6	33.0	66.0	31.5	36.8	42.0	47.3	52.5	57.8	63.0
7	38.5	77.0	36.8	42.9	49.0	55.1	61.3	67.4	73.5
8	44.0	88.0	42.0	49.0	56.0	63.0	70.0	77.0	84.0
9	49.5	99.0	47.3	55.1	63.0	70.9	78.8	86.6	94.5
10	55.0	110.0	52.5	61.3	70.0	78.8	87.5	96.3	105.0

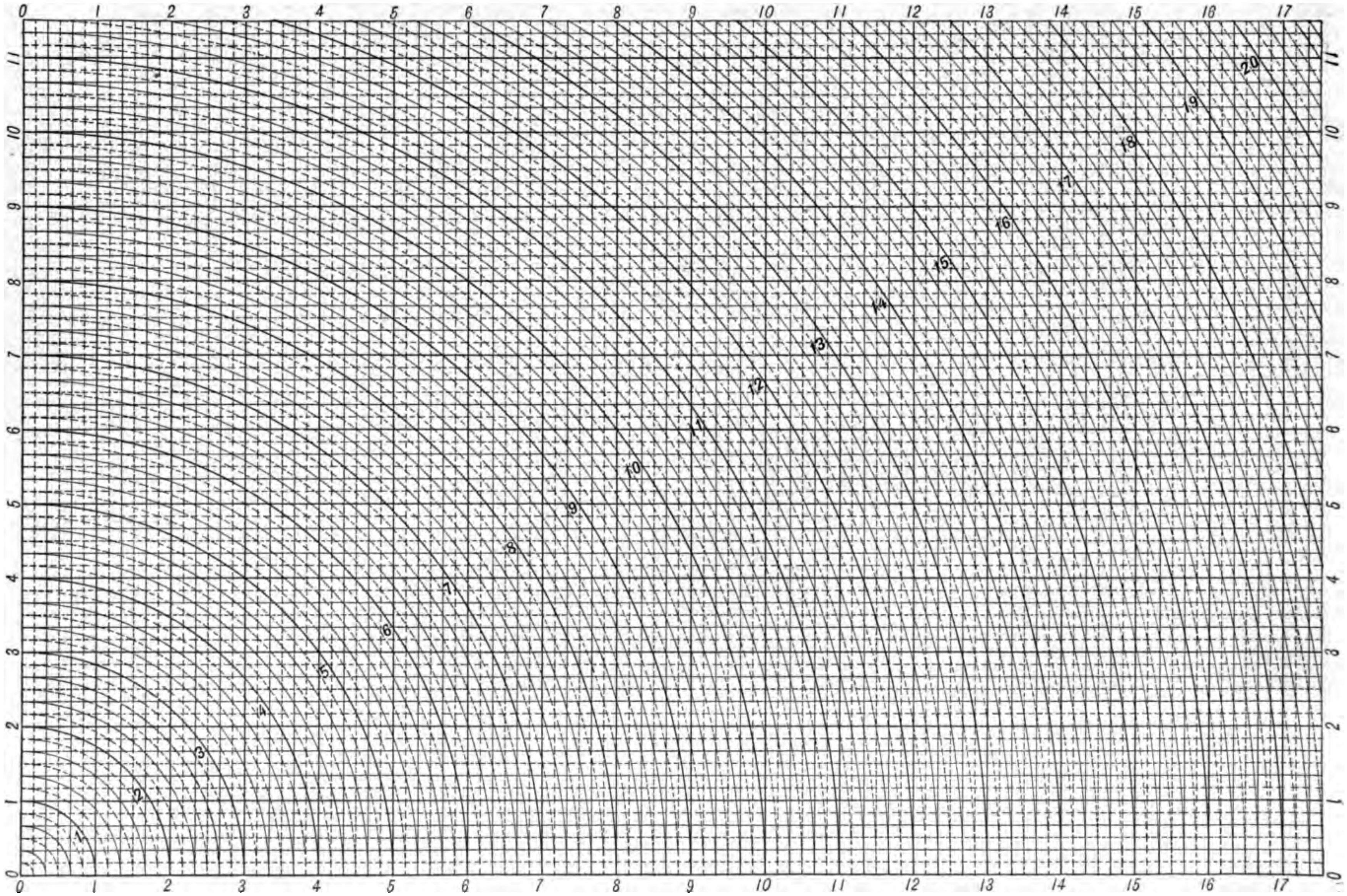
GRAPHIC RESULTANTS—DECIMALS



GRAPHIC RESULTANTS—INCHES AND FRACTIONS



GRAPHIC RESULTANTS-FEET AND INCHES

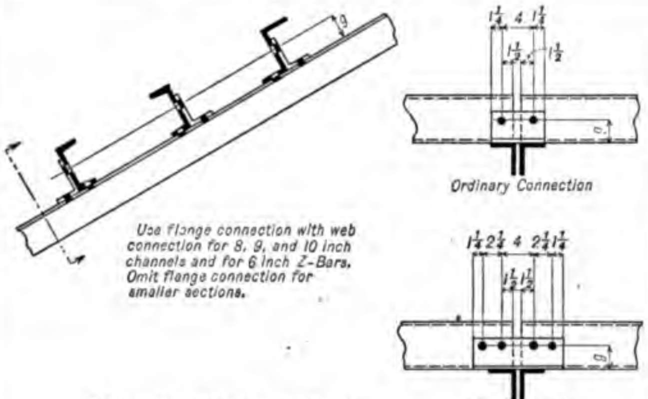


PURLIN CONNECTIONS, LATTICE BARS, AND RODS (AMERICAN BRIDGE COMPANY)

PURLIN CONNECTIONS

Purlin connections are usually bolted

ANGLE PURLINS		
PURLIN DEPTH	CONNECTION ANGLE	GAGE g
3	2½ x 2½	1½
3½	3 x 2½	1¾
4	3 x 2½	2
5	3½ x 2½	2½



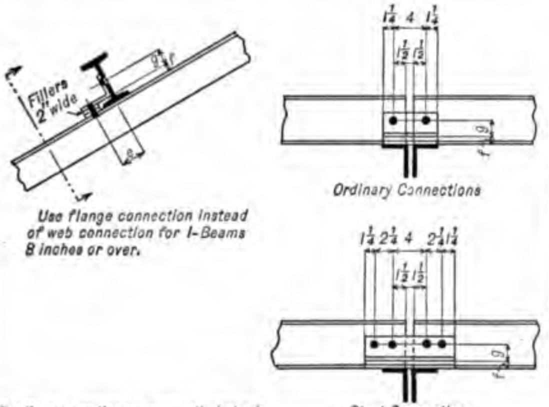
Use flange connection with web connection for 8, 9, and 10 inch channels and for 6 inch Z-Bars. Omit flange connection for smaller sections.

Ordinary Connection

Purlin connections are usually bolted

Strut Connection

CHANNEL AND Z-BAR PURLIN		
PURLIN DEPTH	CONNECTION ANGLE	GAGE g
4	3 x 2½	2
5	3½ x 2½	2½
6	4 x 3	3
7	4 x 3	3
8	4 x 3	3
9	5 x 3½	3½
10	5 x 3½	3½



Use flange connection instead of web connection for I-Beams 8 inches or over.

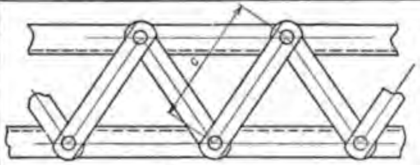
Ordinary Connections

Purlin connections are usually bolted

Strut Connections

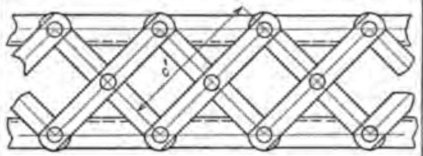
I-BEAM PURLINS				
PURLIN DEPTH	CONNECTION ANGLE	GAGE e	FILLERS f	GAGE g
4	3½ x 2½	2½	¾	1¼
5	4 x 3	2⅞	¾	1¾
6	4 x 3	3	1	2
7	5 x 3½	3⅜	1	2

LATTICE BARS



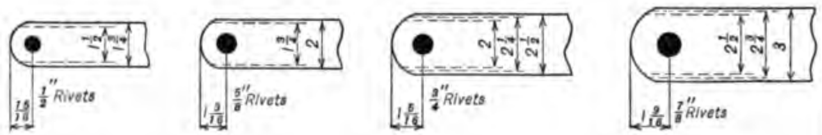
Single Lattice

MAXIMUM DISTANCE c		
THICKNESS t	$c = 40\,t$	$c = 50\,t$
⅝	2'1"	2'7¼"
$\frac{9}{16}$	1'10½"	2'4⅛"
½	1'8"	2'1"
$\frac{7}{16}$	1'5½"	1'9⅞"
⅜	1'3"	1'6¾"
$\frac{5}{16}$	1'0½"	1'3⅝"
¼	10"	1'0½"



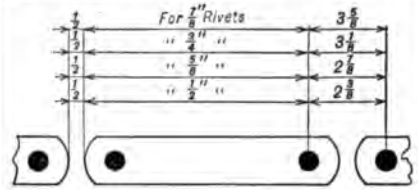
Double Lattice

MAXIMUM DISTANCE c'		
THICKNESS t	$c' = 60\,t$	$c' = 75\,t$
⅝	3'1½"	3'10⅞"
$\frac{9}{16}$	2'9¾"	3'6$\frac{3}{16}$"
½	2'6"	3'1½"
$\frac{7}{16}$	2'2¼"	2'8$\frac{13}{16}$"
⅜	1'10½"	2'4⅛"
$\frac{5}{16}$	1'6¾"	1'11$\frac{7}{16}$"
¼	1'3"	1'6¾"



AREAS AND WEIGHTS OF RODS

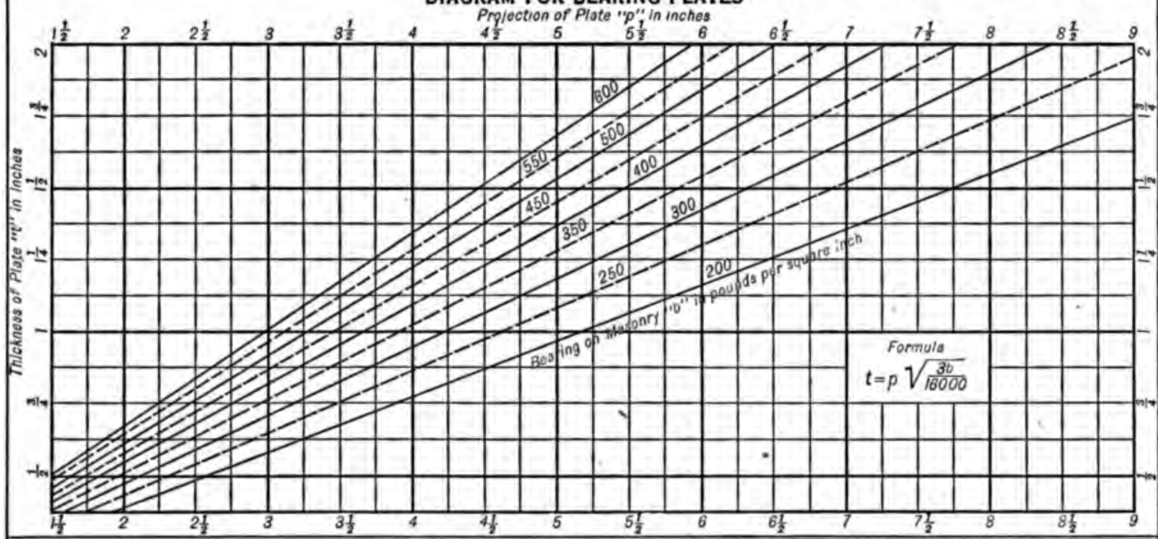
DIAM. IN.	AREA SQ. IN.	WEIGHT PER FT.	UPSET END DIAM.	UPSET END LENGTH	AREA AT ROOT OF THREAD
½	.20	.7	¾ *	4	.30
⅝	.31	1.0	⅞ *	4	.42
¾	.44	1.5	1 *	4	.55
⅞	.60	2.0	1¼ *	4	.89
1	.79	2.7	1⅜	4	1.05
1⅛	.99	3.4	1½	4	1.29
1¼	1.23	4.2	1⅝	4	1.52
1⅜	1.48	5.1	1¾	4	1.74
1½	1.77	6.0	2	4½	2.30
1⅝	2.07	7.1	2⅛	4½	2.65
1¾	2.41	8.2	2¼	5	3.02
1⅞	2.76	9.4	2⅝	5	3.42
2	3.14	10.7	2½	5½	3.72
2⅛	3.55	12.1	2⅝	5½	4.16
2¼	3.98	13.5	2⅞	6	5.11
2⅜	4.43	15.1	3	6	5.43



* SPECIAL

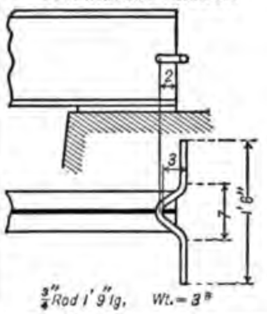
BEARING PLATES, SEPARATORS, ANCHORS AND ROD CONNECTIONS

DIAGRAM FOR BEARING PLATES

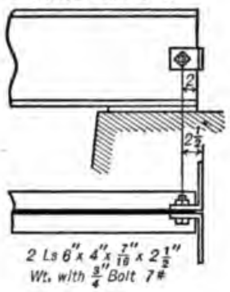


ANCHORS (AMERICAN BRIDGE COMPANY)

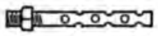
GOVERNMENT ANCHOR



ANGLE ANCHOR



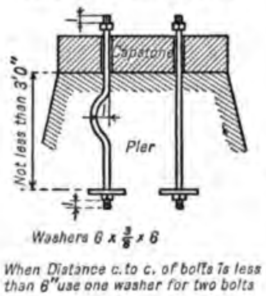
SWEDGE BOLT



WEIGHT INCLUDES NUT

DIAM.	LENGTH	WEIGHT
¾	0′ 9″	1.3
⅞	1′ 0″	2.3
1	1′ 0″	3.1
1¼	1′ 3″	6.1

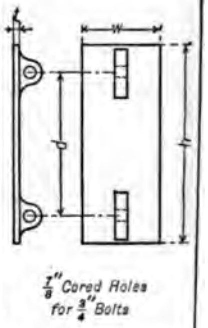
BUILT-IN ANCHOR BOLTS

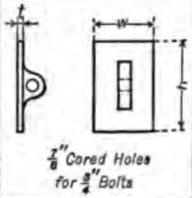


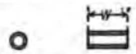
BEAM SEPARATORS (AMERICAN BRIDGE COMPANY)

I-BEAMS			DIMENSIONS				WT. INCL. BOLTS
SIZE	WEIGHT	C. TO C.	w	h	d	t	
IN.	# PER FT.	IN.	IN.	IN.	IN.	IN.	LBS.
24	105-115	8¾	8	20	12	⅝	34
	90-100	8	7¼	20	12	⅝	31
	80-85	8	7½	20	12	⅝	32
20	95-100	8	7	16	12	⅝	25
	75-90	7½	6¾	16	12	⅝	25
	65-70	7	6½	16	12	⅝	24
18	90	8	7	14	9	⅝	23
	80-85	8	7¼	14	9	⅝	24
	75	8	7½	14	9	⅝	24
	65-70	7	6¼	14	9	⅝	21
	55-60	7	6½	14	9	⅝	22
15	75	7	6	11	7½	½	15
	65-70	7	6¼	11	7½	½	15
	55-60	6½	5¾	11	7½	½	14
	42-50	6½	6	11	7½	½	15
12	45-55	6	5¼	8¾	5	½	12
	31½-40	6	5½	8¾	5	½	12
10	35-40	5½	4¾	7½		½	7
	25-30	5½	5	7½		½	8
9	30-35	5	4¼	6½		½	6
	21-25	5	4½	6½		½	6
8	18-25½	4½	4	5½		½	5
7	17½-20	4½	4	5		½	5
	15	4½	4¼	5		½	5
6	14¾-17¼	4	3½	4½		½	5
	12¼	4	3¾	4½		½	5
5	9¾-14¾	3¼	3¼	Use 1" φ Gas Pipe with ¾" Bolt			
4	7½-9½	3¼	3				
3	5½-7½	3	2¾				

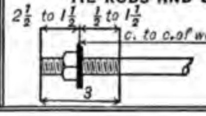
SKETCH





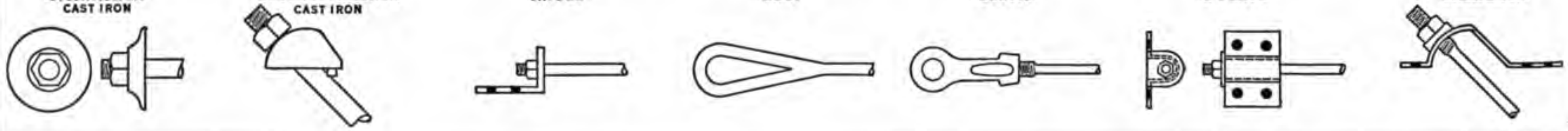


TIE RODS AND SAG RODS (AMERICAN BRIDGE COMPANY)



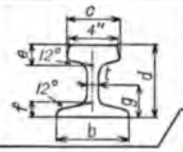
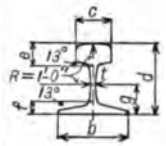
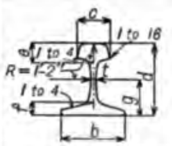
Make Lengths of ¾" Tie Rods and ⅝" Sag Rods multiples of 3" with 3" Thread as shown.

ROD CONNECTIONS (AMERICAN BRIDGE COMPANY)



RAILS, RAIL FASTENINGS, AND UNIT STRESSES FOR STEEL

DIMENSIONS AND PROPERTIES OF RAILS

STANDARD	SKETCH	WEIGHT POUNDS PER YARD	AREA SQ.IN.	SECTION MODULUS s	DEPTH d	FLANGE b	FLANGE f	HEAD c	HEAD e	WEB t	GAGE g	SPLICE BARS FOR CRANE RAILS
CAMBRIA		150	14.7	23.1	6	6	$1\frac{1}{8}$	$4\frac{1}{4}$	$1\frac{3}{4}$	1	$2\frac{11}{16}$	$2\frac{1}{2}$ x $\frac{3}{4}$ x 2'6"
A.S.C.E.		110	10.80	17.2	$6\frac{1}{8}$	$6\frac{1}{8}$	1	$2\frac{7}{8}$	$1\frac{13}{16}$	$\frac{5}{8}$	$2\frac{11}{16}$	3 x $\frac{3}{4}$ x 2'6"
		100	9.84	14.6	$5\frac{3}{4}$	$5\frac{3}{4}$	1	$2\frac{3}{4}$	$1\frac{11}{16}$	$\frac{9}{16}$	$2\frac{1}{2}$	3 x $\frac{5}{8}$ x 2'6"
		95	9.28	13.3	$5\frac{9}{16}$	$5\frac{9}{16}$	$\frac{15}{16}$	$2\frac{11}{16}$	$1\frac{5}{8}$	$\frac{9}{16}$	$2\frac{7}{16}$	$2\frac{3}{4}$ x $\frac{5}{8}$ x 2'6"
		90	8.83	12.2	$5\frac{3}{8}$	$5\frac{3}{8}$	$\frac{15}{16}$	$2\frac{5}{8}$	$1\frac{5}{8}$	$\frac{9}{16}$	$2\frac{3}{8}$	$2\frac{3}{4}$ x $\frac{5}{8}$ x 2'6"
		85	8.33	11.1	$5\frac{3}{16}$	$5\frac{3}{16}$	$\frac{7}{8}$	$2\frac{9}{16}$	$1\frac{9}{16}$	$\frac{9}{16}$	$2\frac{1}{4}$	$2\frac{1}{2}$ x $\frac{5}{8}$ x 2'6"
		80	7.86	10.1	5	5	$\frac{7}{8}$	$2\frac{1}{2}$	$1\frac{1}{2}$	$\frac{9}{16}$	$2\frac{3}{16}$	$2\frac{1}{2}$ x $\frac{1}{2}$ x 2'6"
		75	7.33	9.1	$4\frac{13}{16}$	$4\frac{13}{16}$	$\frac{7}{8}$	$2\frac{1}{2}$	$1\frac{7}{16}$	$\frac{1}{2}$	$2\frac{1}{8}$	$2\frac{1}{4}$ x $\frac{1}{2}$ x 2'6"
		70	6.81	8.2	$4\frac{5}{8}$	$4\frac{5}{8}$	$\frac{13}{16}$	$2\frac{7}{16}$	$1\frac{3}{8}$	$\frac{1}{2}$	$2\frac{1}{16}$	$2\frac{1}{4}$ x $\frac{1}{2}$ x 2'6"
		65	6.33	7.4	$4\frac{7}{16}$	$4\frac{7}{16}$	$\frac{13}{16}$	$2\frac{7}{16}$	$1\frac{5}{16}$	$\frac{1}{2}$	2	$2\frac{1}{4}$ x $\frac{7}{16}$ x 1'6"
		60	5.93	6.6	$4\frac{1}{4}$	$4\frac{1}{4}$	$\frac{3}{4}$	$2\frac{3}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$1\frac{7}{8}$	2 x $\frac{7}{16}$ x 1'6"
		55	5.38	5.7	$4\frac{1}{16}$	$4\frac{1}{16}$	$\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{3}{16}$	$\frac{1}{2}$	$1\frac{13}{16}$	2 x $\frac{3}{8}$ x 1'6"
		50	4.87	5.0	$3\frac{7}{8}$	$3\frac{7}{8}$	$\frac{11}{16}$	$2\frac{1}{8}$	$1\frac{1}{8}$	$\frac{7}{16}$	$1\frac{3}{4}$	2 x $\frac{3}{8}$ x 1'6"
		45	4.40	4.3	$3\frac{11}{16}$	$3\frac{11}{16}$	$\frac{11}{16}$	2	$1\frac{1}{16}$	$\frac{7}{16}$	$1\frac{5}{8}$	$1\frac{3}{4}$ x $\frac{3}{8}$ x 1'6"
		40	3.94	3.6	$3\frac{1}{2}$	$3\frac{1}{2}$	$\frac{5}{8}$	$1\frac{7}{8}$	1	$\frac{3}{8}$	$1\frac{9}{16}$	$1\frac{3}{4}$ x $\frac{5}{16}$ x 1'6"
		35	3.44	3.0	$3\frac{5}{16}$	$3\frac{5}{16}$	$\frac{9}{16}$	$1\frac{3}{4}$	$\frac{15}{16}$	$\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{1}{2}$ x $\frac{5}{16}$ x 1'3"
		30	3.00	2.5	$3\frac{1}{8}$	$3\frac{1}{8}$	$\frac{9}{16}$	$1\frac{11}{16}$	$\frac{7}{8}$	$\frac{5}{16}$	$1\frac{3}{8}$	$1\frac{1}{2}$ x $\frac{1}{4}$ x 1'3"
		25	2.39	1.8	$2\frac{3}{4}$	$2\frac{3}{4}$	$\frac{1}{2}$	$1\frac{1}{2}$	$\frac{13}{16}$	$\frac{5}{16}$	$1\frac{1}{4}$	$1\frac{1}{4}$ x $\frac{1}{4}$ x 1'3"
A.R.E.A.		120	11.85	18.9	$6\frac{1}{2}$	$5\frac{3}{4}$	$1\frac{1}{16}$	$2\frac{5}{8}$	$1\frac{13}{16}$	$\frac{5}{8}$	$3\frac{1}{4}$	3 x $\frac{3}{4}$ x 2'6"
		110	10.82	16.7	$6\frac{1}{4}$	$5\frac{1}{2}$	$1\frac{1}{8}$	$2\frac{13}{16}$	$1\frac{3}{4}$	$\frac{5}{8}$	$3\frac{1}{8}$	3 x $\frac{3}{4}$ x 2'6"
		100	9.95	15.1	6	$5\frac{1}{2}$	$1\frac{1}{16}$	$2\frac{11}{16}$	$1\frac{11}{16}$	$\frac{9}{16}$	3	$2\frac{3}{4}$ x $\frac{5}{8}$ x 2'6"
		90	8.82	12.6	$5\frac{5}{8}$	$5\frac{1}{8}$	1	$2\frac{9}{16}$	$1\frac{1}{2}$	$\frac{9}{16}$	$2\frac{15}{16}$	$2\frac{1}{2}$ x $\frac{5}{8}$ x 2'6"
A.R.A.		100	9.84	15.1	6	$5\frac{1}{2}$	$1\frac{1}{16}$	$2\frac{3}{4}$	$1\frac{11}{16}$	$\frac{9}{16}$	$2\frac{13}{16}$	3 x $\frac{3}{4}$ x 2'6"
		90	8.82	12.5	$5\frac{5}{8}$	$5\frac{1}{8}$	1	$2\frac{9}{16}$	$1\frac{1}{2}$	$\frac{9}{16}$	$2\frac{13}{16}$	$2\frac{3}{4}$ x $\frac{5}{8}$ x 2'6"
		80	7.86	10.2	$5\frac{1}{8}$	$4\frac{5}{8}$	1	$2\frac{1}{2}$	$1\frac{7}{16}$	$\frac{1}{2}$	$2\frac{9}{16}$	$2\frac{1}{2}$ x $\frac{5}{8}$ x 2'6"
		70	6.82	8.3	$4\frac{3}{4}$	$4\frac{1}{4}$	$\frac{15}{16}$	$2\frac{3}{8}$	$1\frac{3}{8}$	$\frac{1}{2}$	$2\frac{7}{16}$	2 x $\frac{1}{2}$ x 2'6"
		60	5.86	6.5	$4\frac{1}{4}$	4	$\frac{13}{16}$	$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{1}{2}$	$2\frac{1}{4}$	2 x $\frac{7}{16}$ x 1'6"

UNIT STRESSES FOR STRUCTURAL STEEL

POUNDS PER SQUARE INCH

RECOMMENDED BY THE AMERICAN RAILWAY ENGINEERING ASSOCIATION

16,000 = AXIAL TENSION, ON NET SECTION.

$16{,}000 - 70\frac{l}{r}$, WITH A MAXIMUM VALUE OF 14,000 = AXIAL COMPRESSION, ON GROSS SECTION, WHERE l IS THE LENGTH OF THE MEMBER IN INCHES AND r IS THE LEAST RADIUS OF GYRATION.

16,000 = DIRECT COMPRESSION, ON STEEL CASTINGS.

16,000 = BENDING, ON EXTREME FIBERS OF ROLLED SHAPES, BUILT SECTIONS, GIRDERS, AND STEEL CASTINGS—NET SECTION.

24,000 = BENDING, ON EXTREME FIBERS OF PINS.

12,000 = SHEARING, ON PINS AND SHOP DRIVEN RIVETS.

10,000 = SHEARING, ON TURNED BOLTS AND FIELD DRIVEN RIVETS.

10,000 = SHEARING, ON PLATE GIRDER WEBS—GROSS SECTION.

24,000 = BEARING, FOR PINS AND SHOP DRIVEN RIVETS.

20,000 = BEARING, FOR TURNED BOLTS AND FIELD DRIVEN RIVETS.

600 = BEARING ON MASONRY.

$600d$ = BEARING ON EXPANSION ROLLERS PER LINEAL INCH WHERE d IS THE DIAMETER OF THE ROLLER.

HOOK BOLTS

FOR CONNECTION TO I-BEAMS

5/8" BOLTS FOR 25-35# RAILS

3/4" " " 40-55" "

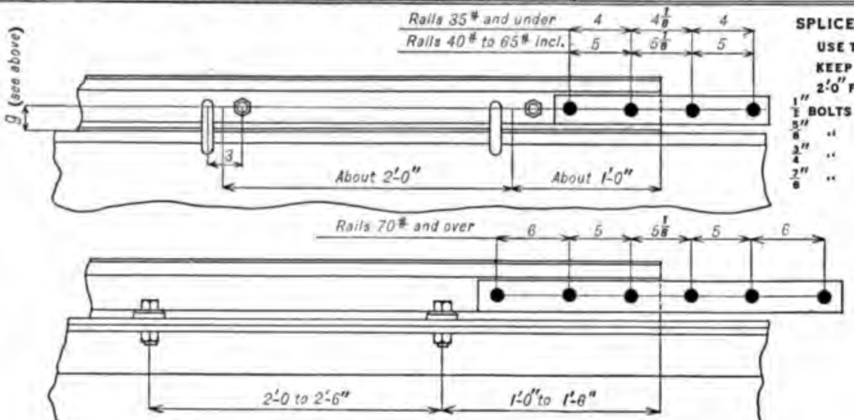
7/8" " " 60-100" "

RAIL CLAMPS

FOR CONNECTION TO GIRDERS

C.I. CLAMPS WITH ECCENTRIC WASHERS FOR ADJUSTMENT.

THE BOLTS ARE OF THE SAME DIAMETER AS THE RIVETS IN THE GIRDER.



SPLICE BARS FOR CRANE RAILS

USE TWO BARS AS SHOWN ABOVE

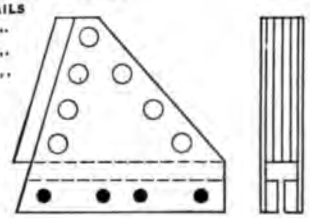
KEEP THE RAIL SPLICES AT LEAST 2'-0" FROM THE JUNCTION OF GIRDERS.

1/2" BOLTS FOR 25# RAILS

5/8" " " 30-35 " "

3/4" " " 40-65 " "

7/8" " " 90-100" "



CRANE STOP

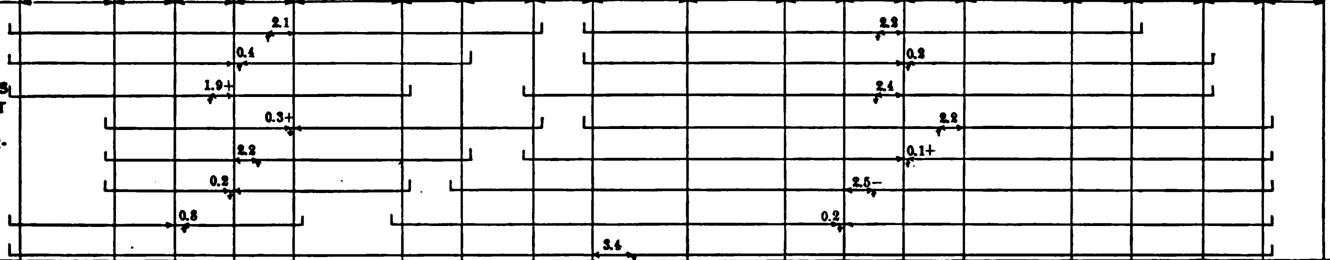
AMERICAN BRIDGE CO.

SHEAR AND MOMENT TABLE FOR COOPER'S E60 LOADING.

ALL LOADS, SHEARS, AND MOMENTS ARE FOR EACH RAIL. HORIZONTAL DISTANCES ARE PLOTTED TO THE SCALE 1 IN.=16 FT. THE VALUES GIVEN IN THE TABLE ARE FOR COOPER'S CONVENTIONAL E60 LOADING. VALUES FOR COOPER'S OTHER LOADINGS MAY BE OBTAINED FROM THOSE IN THE TABLE BY PROPORTION, AS FIVE-SIXTHS FOR E50, OR TWO-THIRDS FOR E40.

	FIRST ENGINE									SECOND ENGINE									TRAIN
	PILOT	DRIVERS				TENDER				PILOT	DRIVERS				TENDER				UNIFORM LOAD
WHEEL (No.)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
WHEEL LOADS ON EACH RAIL IN THOUSANDS OF POUNDS	15.0	30.0	30.0	30.0	30.0	19.5	19.5	19.5	19.5	15.0	30.0	30.0	30.0	30.0	19.5	19.5	19.5	19.5	3.0 PER LIN. FT.
DIST. C. TO C. WHEELS IN FEET	8	5	5	5	9	5	6	5	8	8	5	5	5	9	5	6	5	5	

DISTANCES TO THE CENTERS OF GRAVITY OF DIFFERENT COMBINATIONS OF LOADS, MEASURED FROM THE NEAREST LOADS.



EACH VALUE PRINTED VERTICALLY IS THE DISTANCE FROM THE ZIG-ZAG LINE TO THE LINE NEAREST THE VALUE.

SHEARS — EACH VALUE ON THE LEFT OF ANY VERTICAL OF THE ZIG-ZAG LINE IS THE SUM, IN THOUSANDS OF POUNDS, OF THE LOAD DIRECTLY OVER THIS VERTICAL, OF THE LOAD OVER THE FIRST LINE AT THE LEFT OF THE VALUE, AND OF ALL LOADS BETWEEN THESE TWO.

SHEARS — EACH VALUE ON THE RIGHT OF ANY VERTICAL OF THE ZIG-ZAG LINE IS THE SUM, IN THOUSANDS OF POUNDS, OF THE LOAD DIRECTLY OVER THIS VERTICAL, OF THE LOAD OVER THE FIRST LINE AT THE RIGHT OF THE VALUE, AND OF ALL LOADS BETWEEN THESE TWO.

1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14	14–15	15–16	16–17	17–18	18–
109	101	96	91	86	77	72	68	61	53	45	40	35	30	21	16	10	5
104 428	96 411	91 381	86 351	81 321	72 291	67 271.5	61 252	56 232.5	48 213	40 198	35 168	30 138	25 108	16 78	11 58.5	5 39	
99 406.5	91 391.5	86 361.5	81 331.5	76 301.5	67 271.5	62 252	56 232.5	51 213	43 193.5	35 178.5	30 148.5	25 118.5	20 88.5	11 58.5	6 39	39 5	
93 387	85 372	80 342	75 312	70 282	61 252	56 232.5	50 213	45 193.5	37 174	29 159	24 129	19 99	14 69	5 39	39 6	58.5	
88 367.5	80 352.5	75 322.5	70 292.5	65 262.5	56 232.5	51 213	45 193.5	40 174	32 154.5	24 139.5	19 109.5	14 79.5	9 49.5	39 5	58.5 11	78 16	
79 348	71 333	66 303	61 273	56 243	47 213	42 193.5	36 174	31 154.5	23 135	15 120	10 90	5 60	49.5 9	69 14	88.5 20	108 25	
74 318	66 303	61 273	56 243	51 213	42 183	37 163.5	31 144	26 124.5	18 105	10 90	5 60	60 5	79.5 14	99 19	118.5 25	138 30	
69 288	61 273	56 243	51 213	46 183	37 153	32 133.5	26 114	21 94.5	13 75	5 60	60 5	90 10	109.5 19	129 24	148.5 30	168 35	
64 258	56 243	51 213	46 183	41 153	32 123	27 103.5	21 84	16 64.5	8 45	60 5	90 10	120 15	139.5 24	159 29	178.5 35	198 40	
56 228	48 213	43 183	38 153	33 123	24 93	19 73.5	13 54	8 34.5	45 8	75 13	105 18	135 23	154.5 32	174 37	193.5 43	213 48	

MOMENTS — EACH VERTICAL OF THE ZIG-ZAG LINE CONTAINS THE POINT OF MOMENTS FOR ALL VALUES ON THE LEFT OF THIS LINE. EACH VALUE IS THE MOMENT, IN THOUSANDS OF POUND-FEET, OF ALL LOADS FROM THE POINT OF MOMENTS TO AND INCLUDING THE LOAD OVER THE FIRST LINE AT THE LEFT OF THE VALUE.

MOMENTS — EACH VERTICAL OF THE ZIG-ZAG LINE CONTAINS THE POINT OF MOMENTS FOR ALL VALUES ON THE RIGHT OF THIS LINE. EACH VALUE IS THE MOMENT, IN THOUSANDS OF POUND-FEET, OF ALL LOADS FROM THE POINT OF MOMENTS TO AND INCLUDING THE LOAD OVER THE FIRST LINE AT THE RIGHT OF THE VALUE.

1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14	14–15	15–16	16–17	17–18	18–
24,500	22,900	19,900	17,000	14,300	11,700	10,200	8,790	7,500	6,310	5,510	4,160	2,960	1,910	1,010	605	293	97.5
22,400	20,900	18,000	15,200	12,700	10,200	8,830	7,530	6,340	5,240	4,520	3,320	2,270	1,370	624	312	97.5	
20,400	18,900	16,200	13,600	11,200	8,880	7,570	6,360	5,270	4,280	3,630	2,580	1,690	932	332	117	97.5	
18,100	16,700	14,100	11,700	9,470	7,370	6,180	5,090	4,110	3,230	2,680	1,810	1,090	518	97.5	117	332	
16,200	14,900	12,500	10,300	8,150	6,200	5,110	4,120	3,240	2,460	1,980	1,260	690	270	97.5	312	624	
13,100	11,900	9,780	7,800	5,970	4,290	3,370	2,550	1,850	1,250	900	450	150	176	449	839	1,330	
11,500	10,400	8,410	6,580	4,900	3,370	2,550	1,830	1,230	720	450	150	150	423	794	1,280	1,870	
10,100	9,030	7,200	5,520	3,990	2,610	1,890	1,260	755	345	150	150	450	821	1,290	1,870	2,560	
8,770	7,810	6,130	4,600	3,220	1,990	1,370	842	432	120	150	450	900	1,370	1,930	2,620	3,400	
6,950	6,110	4,670	3,380	2,240	1,250	780	410	156	240	630	1,170	1,860	2,480	3,210	4,040	4,980	

NOMINAL SIZE	SECTION MODULUS s	ACTUAL SIZE ⅛" SCANT	SECTION MODULUS s	ACTUAL SIZE ¼" SCANT	SECTION MODULUS s	ACTUAL SIZE ⅜" SCANT	SECTION MODULUS s	ACTUAL SIZE ½" SCANT	SECTION MODULUS s	NOMINAL SIZE	AREA OF CROSS SECTION	BOARD MEASURE PER FOOT OF LENGTH	WEIGHT PER FOOT OF LENGTH (4 LBS. PER FT. B. M.)
INCHES	INCHES³	INCHES	INCHES³	INCHES	INCHES³	INCHES	INCHES³	INCHES	INCHES³	INCHES	SQUARE INCHES	FEET B. M.	POUNDS
18×18	972	17⅞×17⅞	952	17¾×17¾	932	17⅝×17⅝	912	17½×17½	893	18×18	324	27.0	108
16×18	864	15⅞×17⅞	845	15¾×17¾	827	15⅝×17⅝	809	15½×17½	791	16×18	288	24.0	96
14×18	756	13⅞×17⅞	739	13¾×17¾	722	13⅝×17⅝	705	13½×17½	689	14×18	252	21.0	84
12×18	648	11⅞×17⅞	632	11¾×17¾	617	11⅝×17⅝	602	11½×17½	587	12×18	216	18.0	72
10×18	540	9⅞×17⅞	526	9¾×17¾	512	9⅝×17⅝	498	9½×17½	485	10×18	180	15.0	60
8×18	432	7⅞×17⅞	419	7¾×17¾	407	7⅝×17⅝	395	7½×17½	383	8×18	144	12.0	48
6×18	324	5⅞×17⅞	313	5¾×17¾	302	5⅝×17⅝	291	5½×17½	281	6×18	108	9.0	36
4×18	216	3⅞×17⅞	206	3¾×17¾	197	3⅝×17⅝	188	3½×17½	179	4×18	72	6.0	24
3×18	162	2⅞×17⅞	153	2¾×17¾	144	2⅝×17⅝	136	2½×17½	128	3×18	54	4.5	18
2½×18	135	2⅜×17⅞	126	2¼×17¾	118	2⅛×17⅝	110	2×17½	102	2½×18	45	3.8	15
2×16	108	1⅞×17⅞	100	1¾×17¾	92	1⅝×17⅝	84	1½×17½	77	2×18	36	3.0	12
16×16	683	15⅞×15⅞	667	15¾×15¾	651	15⅝×15⅝	636	15½×15½	621	16×16	256	21.3	85
14×16	597	13⅞×15⅞	583	13¾×15¾	568	13⅝×15⅝	554	13½×15½	541	14×16	224	18.7	75
12×16	512	11⅞×15⅞	499	11¾×15¾	485	11⅝×15⅝	473	11½×15½	460	12×16	192	16.0	64
10×16	427	9⅞×15⅞	415	9¾×15¾	403	9⅝×15⅝	302	9½×15½	380	10×16	160	13.3	53
8×16	341	7⅞×15⅞	331	7¾×15¾	320	7⅝×15⅝	310	7½×15½	300	8×16	128	10.7	43
6×16	256	5⅞×15⅞	247	5¾×15¾	238	5⅝×15⅝	229	5½×15½	220	6×16	96	8.0	32
4×16	171	3⅞×15⅞	163	3¾×15¾	155	3⅝×15⅝	148	3½×15½	140	4×16	64	5.3	21
3×16	128	2⅞×15⅞	121	2¾×15¾	114	2⅝×15⅝	107	2½×15½	100	3×16	48	4.0	16
2½×16	107	2⅜×15⅞	100	2¼×15¾	93	2⅛×15⅝	86	2×15½	80	2½×18	40	3.3	13
2×16	85	1⅞×15⅞	79	1¾×15¾	72	1⅝×15⅝	66	1½×15½	60	2×16	32	2.7	11
14×14	457	13⅞×13⅞	445	13¾×13¾	433	13⅝×13⅝	421	13½×13½	410	14×14	196	16.3	65
12×14	332	11⅞×13⅞	381	11¾×13¾	370	11⅝×13⅝	360	11½×13½	349	12×14	168	14.0	56
10×14	327	9⅞×13⅞	317	9¾×13¾	307	0⅝×13⅝	298	9½×13½	289	10×14	140	11.7	47
8×14	261	7⅞×13⅞	253	7¾×13¾	244	7⅝×13⅝	236	7½×13½	228	8×14	112	9.3	37
6×14	196	5⅞×13⅞	189	5¾×13¾	181	5⅝×13⅝	174	5½×13½	167	6×14	84	7.0	28
4×14	131	3⅞×13⅞	124	3¾×13¾	118	3⅝×13⅝	112	3½×13½	106	4×14	56	4.7	19
3×14	98	2⅞×13⅞	92	2¾×13¾	87	2⅝×13⅝	81	2½×13½	76	3×14	42	3.5	14
2½×14	82	2⅜×13⅞	76	2¼×13¾	71	2⅛×13⅝	66	2×13½	01	2½×14	35	2.9	12
2×14	65	1⅞×13⅞	60	1¾×13¾	55	1⅝×13⅝	50	1½×13½	46	2×14	28	2.3	9
12×12	288	11⅞×11⅞	279	11¾×11¾	270	11⅝×11⅝	262	11½×11½	253	12×12	144	12.0	48
10×12	240	9⅞×11⅞	232	0¾×11¾	224	0⅝×11⅝	217	9½×11½	209	10×12	120	10.0	40
8×12	192	7⅞×11⅞	185	7¾×11¾	178	7⅝×11⅝	172	7½×11½	105	8×12	96	8.0	32
6×12	144	5⅞×11⅞	138	5¾×11¾	132	5⅝×11⅝	127	5½×11½	121	6×12	72	6.0	24
4×12	96	3⅞×11⅞	91	3¾×11¾	86	3⅝×11⅝	82	3½×11½	77	4×12	48	4.0	16
3×12	72	2⅞×11⅞	68	2¾×11¾	63	2⅝×11⅝	59	2½×11½	55	3×12	36	3.0	12
2½×12	60	2⅜×11⅞	56	2¼×11¾	52	2⅛×11⅝	48	2×11½	44	2½×12	30	2.5	10
2×12	48	1⅞×11⅞	44	1¾×11¾	40	1⅝×11⅝	37	1½×11½	33	2×12	24	2.0	8
10×10	167	9⅞×9⅞	160	9¾×0¾	154	9⅝×9⅝	148	9½×0½	143	10×10	100	8.3	33
8×10	133	7⅞×9⅞	128	7¾×0¾	123	7⅝×9⅝	118	7½×9½	113	8×10	80	6.7	27
6×10	100	5⅞×9⅞	95	5¾×9¾	91	5⅝×9⅝	87	5½×9½	83	6×10	60	5.0	20
4×10	67	3⅞×9⅞	63	3¾×9¾	59	3⅝×9⅝	56	3½×9½	53	4×10	40	3.3	13
3×10	50	2⅞×9⅞	47	2¾×0¾	44	2⅝×9⅝	41	2½×9½	38	3×10	30	2.5	10
2½×10	42	2⅜×9⅞	39	2¼×9¾	36	2⅛×9⅝	33	2×0½	30	2½×10	25	2.1	8
2×10	33	1⅞×9⅞	30	1¾×9¾	28	1⅝×9⅝	25	1½×0½	23	2×10	20	1.7	7
8×8	85	7⅞×7⅞	81	7¾×7¾	78	7⅝×7⅝	74	7½×7½	70	8×8	64	5.3	21
6×8	64	5⅞×7⅞	61	5¾×7¾	58	5⅝×7⅝	55	5½×7½	52	6×8	48	4.0	16
4×8	43	3⅞×7⅞	40	3¾×7¾	38	3⅝×7⅝	35	3½×7½	33	4×8	32	2.7	11
3×8	32	2⅞×7⅞	30	2¾×7¾	28	2⅝×7⅝	25	2½×7½	23	3×8	24	2.0	8
2½×8	27	2⅜×7⅞	25	2¼×7¾	23	2⅛×7⅝	21	2×7½	19	2½×8	20	1.7	7
2×8	21	1⅞×7⅞	19	1¾×7¾	18	1⅝×7⅝	16	1½×7½	14	2×8	16	1.3	5
6×6	36	5⅞×5⅞	34	5¾×5¾	32	5⅝×5⅝	30	5½×5½	28	6×6	36	3.0	12
4×6	24	3⅞×5⅞	22	3¾×5¾	21	3⅝×5⅝	19	3½×5½	18	4×6	24	2.0	8
3×6	18	2⅞×5⅞	17	2¾×5¾	15	2⅝×5⅝	14	2½×5½	13	3×6	18	1.5	6
2½×6	15	2⅜×5⅞	14	2¼×5¾	12	2⅛×5⅝	11	2×5½	10	2½×6	15	1.3	5
2×6	12	1⅞×5⅞	11	1¾×5¾	10	1⅝×5⅝	9	1½×5½	8	2×6	12	1 0	4
4×4	11	3⅞×3⅞	10	3¾×3¾	9	3⅝×3⅝	8	3½×3½	7	4×4	16	1.3	5
3×4	8	2⅞×3⅞	7	2¾×3¾	6	2⅝×3⅝	6	2½×3½	5	3×4	12	1.0	4
2×4	5	1⅞×3⅞	5	1¾×3¾	4	1⅝×3⅝	4	1½×3½	3	2×4	8	0.7	3

WORKING UNIT STRESSES FOR STRUCTURAL TIMBER

POUNDS PER SQUARE INCH

RECOMMENDED BY THE AMERICAN RAILWAY ENGINEERING ASSOCIATION (EXCEPT TWO COLUMNS "X")

UNIT STRESSES ARE FOR GREEN TIMBER AND SHOULD BE USED WITHOUT INCREASING THE LIVE LOAD STRESSES FOR IMPACT

Class of Work	Kind of Timber	Tension: Parallel to the Grain X	Bending: Stress in Extreme Fibre	Bending: *Modulus of Elasticity "E"	Shearing: Across the Grain X	Shearing: Parallel to the Grain	Shearing: Longitudinal Shear in Beams	Compression: Crushing Across the Grain	Compression: Crushing Parallel to the Grain	Compression: Short Columns Length under 15 d	Compression: Long Columns Length over 15 d, l = length in inches, d = diameter or least dimension in inches
Railway Bridges and Trestles	Douglas Fir	1200	1200	1,510,000	1000	170	110	310	1200	900	1200 $(1-l/60d)$
	Longleaf Pine	1300	1300	1,610,000	1300	180	120	260	1300	980	1300 $(1-l/60d)$
	Shortleaf Pine	1000	1100	1,480,000	1100	170	130	170	1100	830	1100 $(1-l/60d)$
	White Pine	900	900	1,130,000	500	100	70	150	1000	750	1000 $(1-l/60d)$
	Spruce	1100	1000	1,310,000	1000	150	70	180	1100	830	1100 $(1-l/60d)$
	Norway Pine	800	800	1,190,000	.....	130	100	150	800	600	800 $(1-l/60d)$
	Tamarack	.....	900	1,220,000	.....	170	100	220	1000	750	1000 $(1-l/60d)$
	Western Hemlock	1100	1100	1,480,000	1100	160	100	220	1200	900	1200 $(1-l/60d)$
	Redwood	850	900	800,000	.....	80	.....	150	900	680	900 $(1-l/60d)$
	Bald Cypress	700	900	1,150,000	.....	120	.....	170	1100	830	1100 $(1-l/60d)$
	Red Cedar	800	800	860,000	450	.....	.....	230	900	680	900 $(1-l/60d)$
	White Oak	1100	1100	1,150,000	1000	210	110	450	1300	980	1300 $(1-l/60d)$
Highway Bridges and Trestles	Douglas Fir	1500	1500	1,510,000	1250	210	140	390	1500	1130	1500 $(1-l/60d)$
	Longleaf Pine	1630	1630	1,610,000	1630	230	150	330	1630	1230	1630 $(1-l/60d)$
	Shortleaf Pine	1250	1380	1,480,000	1380	210	160	210	1380	1030	1380 $(1-l/60d)$
	White Pine	1130	1130	1,130,000	630	125	90	190	1250	940	1250 $(1-l/60d)$
	Spruce	1380	1250	1,310,000	1250	190	90	230	1380	1030	1380 $(1-l/60d)$
	Norway Pine	1000	1000	1,190,000	.....	160	125	190	1000	750	1000 $(1-l/60d)$
	Tamarack	.....	1130	1,220,000	.....	210	125	280	1250	940	1250 $(1-l/60d)$
	Western Hemlock	1380	1380	1,480,000	1380	200	125	280	1500	1130	1500 $(1-l/60d)$
	Redwood	1060	1130	800,000	.....	100	.....	190	1130	850	1130 $(1-l/60d)$
	Bald Cypress	880	1130	1,150,000	.....	150	.....	210	1380	1030	1380 $(1-l/60d)$
	Red Cedar	1000	1000	860,000	560	.....	.....	290	1130	850	1130 $(1-l/60d)$
	White Oak	1380	1380	1,150,000	1250	260	140	560	1630	1230	1630 $(1-l/60d)$
Buildings and Similar Structures Free from Impact and Exposure	Douglas Fir	1800	1800	755,000	1500	260	170	470	1800	1350	1800 $(1-l/60d)$
	Longleaf Pine	1950	1950	805,000	1950	270	180	390	1950	1470	1950 $(1-l/60d)$
	Shortleaf Pine	1500	1650	740,000	1650	260	200	260	1650	1250	1650 $(1-l/60d)$
	White Pine	1350	1350	565,000	750	150	110	230	1500	1130	1500 $(1-l/60d)$
	Spruce	1650	1500	655,000	1500	230	110	270	1650	1250	1650 $(1-l/60d)$
	Norway Pine	1200	1200	595,000	.....	200	150	230	1200	900	1200 $(1-l/60d)$
	Tamarack	.....	1350	610,000	.....	260	150	330	1500	1130	1500 $(1-l/60d)$
	Western Hemlock	1650	1650	740,000	1650	240	150	330	1800	1350	1800 $(1-l/60d)$
	Redwood	1280	1350	400,000	.....	120	.....	230	1350	1020	1350 $(1-l/60d)$
	Bald Cypress	1050	1350	575,000	.....	180	.....	260	1650	1250	1650 $(1-l/60d)$
	Red Cedar	1200	1200	430,000	680	.....	.....	350	1350	1020	1350 $(1-l/60d)$
	White Oak	1650	1650	575,000*	1500	320	170	680	1950	1470	1950 $(1-l/60d)$

*USE THE VALUES OF "E" GIVEN IN THE BOTTOM TABLE FOR FINDING THE DEFLECTION OF BEAMS UNDER QUIESCENT OR LONG CONTINUED LOADS. USE THE VALUES IN THE UPPER TABLES WHEN THE LOADS ARE SUDDENLY APPLIED.

MOMENTS OF INERTIA OF RECTANGLES

INCHES^4

NEUTRAL AXIS

MOMENTS OF INERTIA OF WIDER RECTANGLES MAY BE FOUND BY DIRECT PROPORTION

WIDTH OF RECTANGLE (PARALLEL TO NEUTRAL AXIS) IN INCHES

Depth in Inches	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.7
3	0.6	0.7	0.8	1.0	1.1	1.3	1.4	1.6	1.7	1.8	2.0	2.1	2.3
4	1.3	1.7	2.0	2.3	2.7	3.0	3.3	3.7	4.0	4.3	4.7	5.0	5.3
5	2.6	3.3	3.9	4.6	5.2	5.9	6.5	7.2	7.8	8.5	9.1	9.8	10.4
6	4.5	5.6	6.8	7.9	9.0	10.1	11.3	12.4	13.5	14.6	15.8	16.9	18.0
7	7.1	8.9	10.7	12.5	14.3	16.1	17.9	19.7	21.4	23.2	25.0	26.8	28.6
8	10.7	13.3	16.0	18.7	21.3	24.0	26.7	29.3	32.0	34.7	37.3	40.0	42.7
9	15.2	19.0	22.8	26.6	30.4	34.2	38.0	41.8	45.6	49.4	53.2	57.0	60.8
10	20.8	26.0	31.3	36.5	41.7	46.9	52.1	57.3	62.5	67.7	72.9	78.1	83.3
11	27.7	34.7	41.6	48.5	55.5	62.4	69.3	76.3	83.2	90.1	97.1	104.0	110.9
12	36.0	45.0	54.0	63.0	72.0	81.0	90.0	99.0	108.0	117.0	126.0	135.0	144.0
13	45.8	57.2	68.7	80.1	91.5	103.0	114.4	125.9	137.3	148.8	160.2	171.6	183.1
14	57.2	71.5	85.8	100.0	114.3	128.6	142.9	157.2	171.5	185.8	200.1	214.4	228.7
15	70.3	87.9	105.5	123.1	140.6	158.2	175.8	193.4	210.9	228.5	246.1	263.7	281.3
16	85.3	106.7	128.0	149.3	170.7	192.0	213.3	234.7	256.0	277.3	298.7	320.0	341.3
17	102.4	127.9	153.5	179.1	204.7	230.3	255.9	281.5	307.1	332.7	358.2	383.8	409.4
18	121.5	151.9	182.3	212.6	243.0	273.4	303.8	334.1	364.5	394.9	425.3	455.6	486.0
19	142.9	178.6	214.3	250.1	285.8	321.5	357.2	393.0	428.7	464.4	500.1	535.9	571.6
20	166.7	208.3	250.0	291.7	333.3	375.0	416.7	458.3	500.0	541.7	583.3	625.0	666.7
21	192.9	241.2	289.4	337.6	385.9	434.1	482.3	530.6	578.8	627.0	675.3	723.5	771.8
22	221.8	277.3	332.8	388.2	443.7	499.1	554.6	610.0	665.5	721.0	776.4	831.9	887.3
23	253.5	316.8	380.2	443.6	507.0	570.3	633.7	697.1	760.4	823.8	887.2	950.6	1,014
24	288.0	360.0	432.0	504.0	576.0	648.0	720.0	792.0	864.0	936.0	1,008	1,080	1,152
25	325.5	406.9	488.3	569.7	651.0	732.4	813.8	895.2	976.6	1,058	1,139	1,221	1,302
26	366.2	457.7	549.3	640.8	732.3	823.9	915.4	1,007	1,099	1,190	1,282	1,373	1,465
27	410.1	512.6	615.1	717.6	820.1	922.6	1,025	1,128	1,230	1,333	1,435	1,538	1,640
28	457.3	571.7	686.0	800.3	914.7	1,029	1,143	1,258	1,372	1,486	1,601	1,715	1,829
29	508.1	635.1	762.2	889.2	1016	1,143	1,270	1,397	1,524	1,651	1,778	1,905	2,032
30	562.5	703.1	843.8	984.4	1125	1,266	1,406	1,547	1,688	1,828	1,969	2,109	2,250
32	682.7	853.3	1024	1195	1365	1,536	1,707	1,877	2,048	2,219	2,389	2,560	2,731
34	818.8	1024	1228	1433	1638	1,842	2,047	2,252	2,457	2,661	2,866	3,071	3,275
36	972.0	1215	1458	1701	1944	2,187	2,430	2,673	2,916	3,159	3,402	3,645	3,888
38	1143	1429	1715	2001	2286	2,572	2,858	3,144	3,430	3,715	4,001	4,287	4,573
40	1333	1667	2000	2333	2667	3,000	3,333	3,667	4,000	4,333	4,667	5,000	5,333
42	1544	1929	2315	2701	3087	3,473	3,859	4,245	4,631	5,016	5,402	5,788	6,174
44	1775	2218	2662	3106	3549	3,993	4,437	4,880	5,324	5,768	6,211	6,655	7,099
46	2028	2535	3042	3549	4056	4,563	5,070	5,577	6,084	6,590	7,097	7,604	8,111
48	2304	2880	3456	4032	4608	5,184	5,760	6,336	6,912	7,488	8,064	8,640	9,216
50	2604	3255	3906	4557	5208	5,859	6,510	7,161	7,813	8,464	9,115	9,766	10,420
52	2929	3662	4394	5126	5859	6,591	7,323	8,056	8,788	9,520	10,250	10,990	11,720
54	3281	4101	4921	5741	6561	7,381	8,201	9,021	9,842	10,660	11,480	12,300	13,120
56	3659	4573	5488	6403	7317	8,232	9,147	10,060	10,980	11,890	12,810	13,720	14,630
58	4065	5081	6097	7113	8130	9,146	10,160	11,180	12,190	13,210	14,230	15,240	16,260
60	4500	5625	6750	7875	9000	10,130	11,250	12,380	13,500	14,620	15,750	16,880	18,000

AREAS OF PLATES IN SQUARE INCHES

THICKNESS IN INCHES

3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	1 3/16	1 1/4	1 5/16	1 3/8	1 7/16	1 1/2	1 9/16	1 5/8	1 11/16	1 3/4	1 13/16	1 7/8	1 15/16	2
1.13	1.31	1.50	1.69	1.88	2.06	2.25	2.44	2.63	2.81	3.00	3.19	3.38	3.56	3.75	3.94	4.13	4.31	4.50	4.69	4.88	5.06	5.25	5.44	5.63	5.81	6.00
1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75	5.00	5.25	5.50	5.75	6.00	6.25	6.50	6.75	7.00	7.25	7.50	7.75	8.00
1.88	2.19	2.50	2.81	3.13	3.44	3.75	4.06	4.38	4.69	5.00	5.31	5.63	5.94	6.25	6.56	6.88	7.19	7.50	7.81	8.13	8.44	8.75	9.06	9.38	9.69	10.00
2.25	2.63	3.00	3.38	3.75	4.13	4.50	4.88	5.25	5.63	6.00	6.38	6.75	7.13	7.50	7.88	8.25	8.63	9.00	9.38	9.75	10.13	10.50	10.88	11.25	11.63	12.00
2.63	3.06	3.50	3.94	4.38	4.81	5.25	5.69	6.13	6.56	7.00	7.44	7.88	8.31	8.75	9.19	9.63	10.06	10.50	10.94	11.38	11.81	12.25	12.69	13.13	13.56	14.00
3.00	3.50	4.00	4.50	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00	12.50	13.00	13.50	14.00	14.50	15.00	15.50	16.00
3.38	3.94	4.50	5.06	5.63	6.19	6.75	7.31	7.88	8.44	9.00	9.56	10.13	10.69	11.25	11.81	12.38	12.94	13.50	14.06	14.63	15.19	15.75	16.31	16.88	17.44	18.00
3.75	4.38	5.00	5.63	6.25	6.88	7.50	8.13	8.75	9.38	10.00	10.63	11.25	11.88	12.50	13.13	13.75	14.38	15.00	15.63	16.25	16.88	17.50	18.13	18.75	19.38	20.00
3.84	4.48	5.13	5.77	6.41	7.05	7.69	8.33	8.97	9.61	10.25	10.89	11.53	12.17	12.81	13.45	14.09	14.73	15.38	16.02	16.66	17.30	17.94	18.58	19.22	19.86	20.50
3.94	4.59	5.25	5.91	6.56	7.22	7.88	8.53	9.19	9.84	10.50	11.16	11.81	12.47	13.13	13.78	14.44	15.09	15.75	16.41	17.06	17.72	18.38	19.03	19.69	20.34	21.00
4.03	4.70	5.38	6.05	6.72	7.39	8.06	8.74	9.41	10.08	10.75	11.42	12.09	12.77	13.44	14.11	14.78	15.45	16.13	16.80	17.47	18.14	18.81	19.48	20.16	20.83	21.50
4.13	4.81	5.50	6.19	6.88	7.56	8.25	8.94	9.63	10.31	11.00	11.69	12.38	13.06	13.75	14.44	15.13	15.81	16.50	17.19	17.88	18.56	19.25	19.94	20.63	21.31	22.00
4.22	4.92	5.63	6.33	7.03	7.73	8.44	9.14	9.84	10.55	11.25	11.95	12.66	13.36	14.06	14.77	15.47	16.17	16.88	17.58	18.28	18.98	19.69	20.39	21.09	21.80	22.50
4.31	5.03	5.75	6.47	7.19	7.91	8.63	9.34	10.06	10.78	11.50	12.22	12.94	13.66	14.38	15.09	15.81	16.53	17.25	17.97	18.69	19.41	20.13	20.84	21.56	22.28	23.00
4.41	5.14	5.88	6.61	7.34	8.08	8.81	9.55	10.28	11.02	11.75	12.48	13.22	13.95	14.69	15.42	16.16	16.89	17.63	18.36	19.09	19.83	20.56	21.30	22.03	22.77	23.50
4.50	5.25	6.00	6.75	7.50	8.25	9.00	9.75	10.50	11.25	12.00	12.75	13.50	14.25	15.00	15.75	16.50	17.25	18.00	18.75	19.50	20.25	21.00	21.75	22.50	23.25	24.00
4.59	5.36	6.13	6.89	7.66	8.42	9.19	9.95	10.72	11.48	12.25	13.02	13.78	14.55	15.31	16.08	16.84	17.61	18.38	19.14	19.91	20.67	21.44	22.20	22.97	23.73	24.50
4.69	5.47	6.25	7.03	7.81	8.59	9.38	10.16	10.94	11.72	12.50	13.28	14.06	14.84	15.63	16.41	17.19	17.97	18.75	19.53	20.31	21.09	21.88	22.66	23.44	24.22	25.00
4.78	5.58	6.38	7.17	7.97	8.77	9.56	10.36	11.16	11.95	12.75	13.55	14.34	15.14	15.94	16.73	17.53	18.33	19.13	19.92	20.72	21.52	22.31	23.11	23.91	24.70	25.50
4.88	5.69	6.50	7.31	8.13	8.94	9.75	10.56	11.38	12.19	13.00	13.81	14.63	15.44	16.25	17.06	17.88	18.69	19.50	20.31	21.13	21.94	22.75	23.56	24.38	25.19	26.00
4.97	5.80	6.63	7.45	8.28	9.11	9.94	10.77	11.59	12.42	13.25	14.08	14.91	15.73	16.56	17.39	18.22	19.05	19.88	20.70	21.53	22.36	23.19	24.02	24.84	25.67	26.50
5.06	5.91	6.75	7.59	8.44	9.28	10.13	10.97	11.81	12.66	13.50	14.34	15.19	16.03	16.88	17.72	18.56	19.41	20.25	21.09	21.94	22.78	23.63	24.47	25.31	26.16	27.00
5.16	6.02	6.88	7.73	8.59	9.45	10.31	11.17	12.03	12.89	13.75	14.61	15.47	16.33	17.19	18.05	18.91	19.77	20.63	21.48	22.34	23.20	24.06	24.92	25.78	26.61	27.50
5.25	6.13	7.00	7.88	8.75	9.63	10.50	11.38	12.25	13.13	14.00	14.88	15.75	16.63	17.50	18.38	19.25	20.13	21.00	21.88	22.75	23.63	24.50	25.38	26.25	27.13	28.00
5.34	6.23	7.13	8.02	8.91	9.80	10.69	11.58	12.47	13.36	14.25	15.14	16.03	16.92	17.81	18.70	19.59	20.48	21.38	22.27	23.16	24.05	24.94	25.83	26.72	27.61	28.50
5.44	6.34	7.25	8.16	9.06	9.97	10.88	11.78	12.69	13.59	14.50	15.41	16.31	17.22	18.13	19.03	19.94	20.84	21.75	22.66	23.56	24.47	25.38	26.28	27.19	28.09	29.00
5.53	6.45	7.38	8.30	9.22	10.14	11.06	11.98	12.91	13.83	14.75	15.67	16.59	17.52	18.44	19.36	20.28	21.20	22.13	23.05	23.97	24.89	25.81	26.73	27.66	28.58	29.50
5.63	6.56	7.50	8.44	9.38	10.31	11.25	12.19	13.13	14.06	15.00	15.94	16.88	17.81	18.75	19.69	20.63	21.56	22.50	23.44	24.38	25.31	26.25	27.19	28.13	29.06	30.00
5.72	6.67	7.63	8.58	9.53	10.48	11.44	12.39	13.34	14.30	15.25	16.20	17.16	18.11	19.06	20.02	20.97	21.92	22.88	23.83	24.78	25.73	26.69	27.64	28.59	29.55	30.50
5.81	6.78	7.75	8.72	9.69	10.66	11.63	12.59	13.56	14.53	15.50	16.47	17.44	18.41	19.38	20.34	21.31	22.28	23.25	24.22	25.19	26.16	27.13	28.09	29.06	30.03	31.00
5.91	6.89	7.88	8.86	9.84	10.83	11.81	12.80	13.78	14.77	15.75	16.73	17.72	18.70	19.69	20.67	21.66	22.64	23.63	24.61	25.59	26.58	27.56	28.55	29.53	30.52	31.50
6.00	7.00	8.00	9.00	10.00	11.00	12.00	13.00	14.00	15.00	16.00	17.00	18.00	19.00	20.00	21.00	22.00	23.00	24.00	25.00	26.00	27.00	28.00	29.00	30.00	31.00	32.00
6.09	7.11	8.13	9.14	10.16	11.17	12.19	13.20	14.22	15.23	16.25	17.27	18.28	19.30	20.31	21.33	22.34	23.36	24.38	25.39	26.41	27.42	28.44	29.45	30.47	31.48	32.50
6.19	7.22	8.25	9.28	10.31	11.34	12.38	13.41	14.44	15.47	16.50	17.53	18.56	19.59	20.63	21.66	22.69	23.72	24.75	25.78	26.81	27.84	28.88	29.91	30.94	31.97	33.00
6.28	7.33	8.38	9.42	10.47	11.52	12.56	13.61	14.66	15.70	16.75	17.80	18.84	19.89	20.94	21.98	23.03	24.08	25.13	26.17	27.22	28.27	29.31	30.36	31.41	32.45	33.50
6.38	7.44	8.50	9.56	10.63	11.69	12.75	13.81	14.88	15.94	17.00	18.06	19.13	20.19	21.25	22.31	23.38	24.44	25.50	26.56	27.63	28.69	29.75	30.81	31.88	32.94	34.00
6.75	7.88	9.00	10.13	11.25	12.38	13.50	14.63	15.75	16.88	18.00	19.13	20.25	21.38	22.50	23.63	24.75	25.88	27.00	28.13	29.25	30.38	31.50	32.63	33.75	34.88	36.00
7.13	8.31	9.50	10.69	11.88	13.06	14.25	15.44	16.63	17.81	19.00	20.19	21.38	22.56	23.75	24.94	26.13	27.31	28.50	29.69	30.88	32.06	33.25	34.44	35.63	36.81	38.00
7.50	8.75	10.00	11.25	12.50	13.75	15.00	16.25	17.50	18.75	20.00	21.25	22.50	23.75	25.00	26.25	27.50	28.75	30.00	31.25	32.50	33.75	35.00	36.25	37.50	38.75	40.00
7.88	9.19	10.50	11.81	13.13	14.44	15.75	17.06	18.38	19.69	21.00	22.31	23.63														
8.25	9.63	11.00	12.38	13.75	15.13	16.50	17.88	19.25	20.63	22.00	23.38	24.75														
8.63	10.06	11.50	12.94	14.38	15.81	17.25	18.69	20.13	21.56	23.00	24.44	25.88														
9.00	10.50	12.00	13.50	15.00	16.50	18.00	19.50	21.00	22.50	24.00	25.50	27.00														
9.38	10.94	12.50	14.06	15.63	17.19	18.75	20.31	21.88	23.44	25.00	26.56	28.12														
9.75	11.38	13.00	14.63	16.25	17.88	19.50	21.13	22.75	24.38	26.00	27.63	29.25														
10.13	11.81	13.50	15.19	16.88	18.56	20.25	21.94	23.63	25.31	27.00	28.69	30.38														
10.50	12.25	14.00	15.75	17.50	19.25	21.00	22.75	24.50	26.25	28.00	29.75	31.50														
10.88	12.69	14.50	16.31	18.13	19.94	21.75	23.56	25.38	27.19	29.00	30.81	32.63														
11.25	13.13	15.00	16.88	18.75	20.63	22.50	24.38	26.25	28.13	30.00	31.88	33.75														
12.00	14.00	16.00	18.00	20.00	22.00	24.00	26.00	28.00	30.00	32.00	34.00	36.00														
12.75	14.88	17.00	19.13	21.25	23.38	25.50	27.63	29.75	31.88	34.00	36.13	38.25														
13.50	15.75	18.00	20.25	22.50	24.75	27.00	29.25	31.50	33.75	36.00	38.25	40.50														
14.25	16.63	19.00	21.38	23.75	26.13	28.50	30.88	33.25	35.63	38.00	40.38	42.75														
15.00	17.50	20.00	22.50	25.00	27.50	30.00	32.50	35.00	37.50	40.00	42.50	45.00														
15.75	18.38	21.00	23.63	26.25	28.88	31.50	34.13	36.75	39.38	42.00	43.63	47.25														
16.50	19.25	22.00	24.75	27.50	30.25	33.00	35.75	38.50	41.25	44.00	46.75	49.50														
17.25	20.13	23.00	25.88	28.75	31.63	34.50	37.38	40.25	43.13	46.00	48.88	51.75														
18.00	21.00	24.00	27.00	30.00	33.00	36.00	39.00	42.00	45.00	48.00	51.00	54.00														
18.75	21.88	25.00	28.13	31.25	34.38	37.50	40.63	43.75	46.88	50.00	53.13	56.25														
19.50	22.75	26.00	29.25	32.50	35.75	39.00	42.25	45.50	48.75	52.00	55.25	58.50														
20.25	23.63	27.00	30.38	33.75	37.13	40.50	43.88	47.25	50.63	54.00	57.38	60.75														
21.00	24.50	28.00	31.50	35.00	38.50	42.00	45.50	49.00	52.50	56.00	59.50	63.00														
21.75	25.38	29.00	32.63	36.25	39.88	43.50	47.13	50.75	54.38	58.00	61.63	65.25														
22.50	26.25	30.00	33.75	37.50	41.25	45.00	48.75	52.50	56.25	60.00	63.75	67.50														

AREAS OF PLATES IN SQUARE INCHES

	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
66	16.50	20.63	24.75	28.88	33.00	27.13	41.25	45.38	49.50	53.63	57.75	61.88	66.00
72	18.00	22.50	27.00	31.50	36.00	40.50	45.00	49.50	54.00	58.50	63.00	67.50	72.00
78	19.50	24.38	29.25	34.13	39.00	43.88	48.75	53.63	58.50	63.38	68.25	73.13	78.00
84	21.00	26.25	31.50	36.75	42.00	47.25	52.50	57.75	63.00	68.25	73.50	78.75	84.00
90	22.50	28.13	33.75	39.38	45.00	50.63	56.25	61.88	67.50	73.13	78.75	84.38	90.00
96	24.00	30.00	36.00	42.00	48.00	54.00	60.00	66.00	72.00	78.00	84.00	90.00	96.00
102	25.50	31.88	38.25	44.63	51.00	57.38	63.75	70.13	76.50	82.88	89.25	95.63	102.00
108	27.00	33.75	40.50	47.25	54.00	60.75	67.50	74.25	81.00	87.75	94.50	101.25	108.00
114	28.50	35.63	42.75	49.88	57.00	64.13	71.25	78.38	85.50	92.63	99.75	106.88	114.00
120	30.00	37.50	45.00	52.50	60.00	67.50	75.00	82.50	90.00	97.50	105.00	112.50	120.00

WEIGHTS OF PLATES IN POUNDS PER LINEAL FOOT *

	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
66	56.1	70.1	84.2	98.2	112.2	126.2	140.3	154.3	168.3	182.3	196.4	210.4	224.4
72	61.2	76.5	91.8	107.1	122.4	137.7	153.0	168.3	183.6	198.9	214.2	229.5	244.8
78	66.3	82.9	99.5	116.0	132.6	149.2	165.8	182.3	198.9	215.5	232.1	248.6	265.2
84	71.4	89.3	107.1	125.0	142.8	160.7	178.5	196.4	214.2	232.1	249.9	267.8	285.6
90	76.5	95.6	114.8	133.9	153.0	172.1	191.3	210.4	229.5	248.6	267.8	286.9	306.0
96	81.6	102.0	122.4	142.8	163.2	183.6	204.0	224.4	244.8	265.2	285.6	306.0	326.4
102	86.7	108.4	130.1	151.8	173.4	195.1	216.7	238.4	260.2	281.8	303.5	325.1	346.8
108	91.8	114.7	137.8	160.7	183.7	206.6	229.5	252.4	275.5	298.3	321.3	344.3	367.2
114	96.9	121.1	145.4	169.6	103.8	218.1	242.2	266.4	290.8	314.9	339.2	363.4	387.6
120	102.0	127.5	153.0	178.5	204.0	229.5	255.0	280.5	306.0	331.5	357.0	382.5	408.0

WEIGHTS OF PLATES IN POUNDS PER LINEAL FOOT *

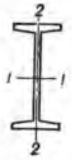
THICKNESS IN INCHES

WIDTH IN INS.	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
1	0.9	1.1	1.3	1.5	1.7	1.9	2.1	2.3	2.6	2.8	3.0	3.2	3.4
2	1.7	2.1	2.6	3.0	3.4	3.8	4.3	4.7	5.1	5.5	6.0	6.4	6.8
3	2.6	3.2	3.8	4.5	5.1	5.7	6.4	7.0	7.7	8.3	8.9	9.6	10.2
4	3.4	4.3	5.1	6.0	6.8	7.7	8.5	9.4	10.2	11.1	11.9	12.8	13.6
5	4.3	5.3	6.4	7.4	8.5	9.6	10.6	11.7	12.8	13.8	14.9	15.9	17.0
6	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4
7	6.0	7.4	8.9	10.4	11.9	13.4	14.9	16.4	17.9	19.3	20.8	22.3	23.8
8	6.8	8.5	10.2	11.9	13.6	15.3	17.0	18.7	20.4	22.1	23.8	25.5	27.2
9	7.7	9.6	11.5	13.4	15.3	17.2	19.1	21.0	23.0	24.9	26.8	28.7	30.6
10	8.5	10.6	12.8	14.9	17.0	19.1	21.3	23.4	25.5	27.6	29.8	31.9	34.0
11	9.4	11.7	14.0	16.4	18.7	21.0	23.4	25.7	28.1	30.4	32.7	35.1	37.4
12	10.2	12.8	15.3	17.9	20.4	23.0	25.5	28.1	30.6	33.2	35.7	38.3	40.8
13	11.1	13.8	16.6	19.3	22.1	24.9	27.6	30.4	33.2	35.9	38.7	41.4	44.2
14	11.9	14.9	17.9	20.8	23.8	26.8	29.7	32.7	35.7	38.7	41.7	44.6	47.6
15	12.8	15.9	19.1	22.3	25.5	28.7	31.9	35.1	38.3	41.4	44.6	47.8	51.0
16	13.6	17.0	20.4	23.8	27.2	30.6	34.0	37.4	40.8	44.2	47.6	51.0	54.4
17	14.5	18.1	21.7	25.3	28.9	32.5	36.1	39.7	43.4	47.0	50.6	54.2	57.8
18	15.3	19.1	23.0	26.8	30.6	34.4	38.3	42.1	45.9	49.7	53.6	57.4	61.2
19	16.2	20.2	24.2	28.3	32.3	36.3	40.4	44.4	48.5	52.5	56.5	60.6	64.6
20	17.0	21.3	25.5	29.8	34.0	38.3	42.5	46.8	51.0	55.3	59.5	63.8	68.0
21	17.9	22.3	26.8	31.2	35.7	40.2	44.6	49.1	53.6	58.0	62.5	66.9	71.4
22	18.7	23.4	28.1	32.7	37.4	42.1	46.8	51.4	56.1	60.8	65.5	70.1	74.8
23	20.0	24.4	29.3	34.2	39.1	44.0	48.9	53.8	58.7	63.5	68.4	73.3	78.2
24	20.4	25.5	30.6	35.7	40.8	45.9	51.0	56.1	61.2	66.3	71.4	76.5	81.6
25	21.3	26.6	31.9	37.2	42.5	47.8	53.1	58.4	63.8	69.1	74.4	79.7	85.0
26	22.1	27.6	33.2	38.7	44.2	49.7	55.3	60.8	66.3	71.8	77.4	82.9	88.4
27	23.0	28.7	34.4	40.2	45.9	51.6	57.4	63.1	68.9	74.6	80.3	86.1	91.8
28	23.8	29.8	35.7	41.7	47.6	53.6	59.5	65.5	71.4	77.4	83.3	89.3	95.2
29	24.7	30.8	37.0	43.1	49.3	55.5	61.6	67.8	74.0	80.1	86.3	92.4	98.6
30	25.5	31.9	38.3	44.6	51.0	57.4	63.8	70.1	76.5	82.9	89.3	95.6	102.0
31	26.4	32.9	39.5	46.1	52.7	59.3	65.9	72.5	79.1	85.6	92.2	98.8	105.4
32	27.2	34.0	40.8	47.6	54.4	61.2	68.0	74.8	81.6	88.4	95.2	102.0	108.8
33	28.1	35.1	42.1	49.1	56.1	63.1	70.1	77.1	84.2	91.2	98.2	105.2	112.2
34	28.9	36.1	43.4	50.6	57.8	65.0	72.3	79.5	86.7	93.9	101.2	108.4	115.6
35	29.8	37.2	44.6	52.1	59.5	66.9	74.4	81.8	89.3	96.7	104.1	111.6	119.0
36	30.6	38.3	45.9	53.6	61.2	68.9	76.5	84.2	91.8	99.5	107.1	114.8	122.4
37	31.5	39.3	47.2	55.0	62.9	70.8	78.6	86.5	94.4	102.2	110.1	117.9	125.8
38	32.3	40.4	48.5	56.5	64.6	72.7	80.8	88.8	96.9	[illegible]	113.1	121.1	129.2
39	33.2	41.4	49.7	58.0	66.3	74.6	82.9	91.2	99.5	107.7	116.0	124.3	132.6
40	34.0	42.5	51.0	59.5	68.0	76.5	85.0	93.5	102.0	110.5	119.0	127.5	136.0
41	34.9	43.6	52.3	61.0	69.7	78.4	87.1	95.8	104.6	113.3	122.0	130.7	139.4
42	35.7	44.6	53.6	62.5	71.4	80.3	89.3	98.2	107.1	116.0	125.0	133.9	142.8
43	36.6	45.7	54.8	64.0	73.1	82.2	91.4	100.5	109.7	118.8	127.9	137.1	146.2
44	37.4	46.8	56.1	65.5	74.8	84.2	93.5	102.9	112.2	121.6	130.9	140.3	149.6
45	38.3	47.8	57.4	66.9	76.5	86.1	95.6	105.2	114.8	124.3	133.9	143.4	153.0
46	39.1	48.9	58.7	68.4	78.2	88.0	97.8	107.5	117.3	127.1	136.9	146.6	156.4
47	40.0	49.9	59.9	69.9	79.9	89.9	99.9	109.9	119.9	129.8	139.8	149.8	159.8
48	40.8	51.0	61.2	71.4	81.6	91.8	102.0	112.2	122.4	132.6	142.8	153.0	163.2
49	41.7	52.1	62.5	72.9	83.3	93.7	104.1	114.5	125.0	135.4	145.8	156.2	166.6
50	42.5	53.1	63.8	74.4	85.0	95.6	106.3	116.9	127.5	138.1	148.8	159.4	170.0
51	43.4	54.2	65.0	75.9	86.7	97.5	108.4	119.2	130.1	140.9	151.7	162.6	173.4
52	44.2	55.3	66.3	77.4	88.4	99.5	110.5	121.6	132.6	143.[illegible]	154.7	165.8	176.8
53	45.1	56.3	67.6	78.8	90.1	101.4	112.6	123.9	135.2	146.4	157.7	168.9	180.2
54	45.9	57.4	68.9	80.3	91.8	103.3	114.8	126.2	137.7	149.2	160.7	172.1	183.6
55	46.8	58.4	70.1	81.8	93.5	105.2	116.9	128.6	140.3	151.9	163.6	175.3	187.0
56	47.6	59.5	71.4	83.3	95.2	107.1	119.0	130.9	142.8	154.7	166.6	178.5	190.4
57	48.5	60.6	72.7	84.8	96.9	109.0	121.1	133.2	145.4	157.5	169.6	181.7	193.8
58	49.3	61.6	74.0	86.3	98.6	110.9	123.3	135.6	147.9	160.2	172.6	184.9	197.2
59	50.2	62.7	75.2	87.8	100.3	112.8	125.4	137.9	150.5	163.0	175.5	188.1	200.6
60	51.0	63.8	76.5	89.3	102.0	114.8	127.5	140.3	153.0	165.8	178.5	191.3	204.0
61	51.9	64.8	77.8	90.7	103.7	116.7	129.6	142.6	155.6	168.5	181.5	194.4	207.4
62	52.7	65.9	79.1	92.2	105.4	118.6	131.8	144.9	158.1	171.3	184.5	197.6	210.8
63	53.6	66.9	80.3	93.7	107.1	120.5	133.9	147.3	160.7	174.0	187.4	200.8	214.2
64	54.4	68.0	81.6	95.2	108.8	122.4	136.0	149.6	163.2	176.8	190.4	204.0	217.6

* WEIGHT = 3.4 × AREA, OR 12 CUBIC INCHES OF STEEL WEIGHS 3.4 POUNDS

IF THE THICKNESS OF THE WEB IS MORE THAN $\frac{1}{64}$ BELOW AN EVEN SIXTEENTH, THE NEXT LOWER SIXTEENTH IS GIVEN IN THE TABLE BELOW

I = THE MOMENT OF INERTIA OF THE CROSS SECTION ABOUT THE AXIS 1-1
s = THE CORRESPONDING SECTION MODULUS ABOUT THE AXIS 1-1
r = THE CORRESPONDING RADIUS OF GYRATION ABOUT THE AXIS 1-1
I_2 = THE MOMENT OF INERTIA OF THE CROSS SECTION ABOUT THE AXIS 2-2
r_2 = THE CORRESPONDING RADIUS OF GYRATION ABOUT THE AXIS 2-2



CARNEGIE I-BEAMS

SIZE IN.	WEIGHT LBS./FT.	WEB IN.		AREA SQ.IN.	I IN.⁴	s IN.³	r IN.	I_2 IN.⁴	r_2 IN.
27	90*	.52	1/2	26.33	2958.3	219.1	10.60	75.3	1.69
24	115	.75	3/4	33.98	2955.5	246.3	9.33	83.2	1.57
	110	.69	11/16	32 48	2883.5	240.3	9.42	81.0	1.58
	105	.63	5/8	30.98	2811.5	234.3	9.53	78.9	1.60
	100	.75	3/4	29.41	2379.6	198.3	9.00	48.6	1.28
	95	.69	11/16	27.94	2309.0	192.4	9.09	47.1	1.30
	90	.63	5/8	26.47	2238.4	186.5	9.20	45.7	1.31
	85	.57	9/16	25.00	2167.8	180.7	9.31	44.4	1.33
	80	.50	1/2	23.32	2087.2	173.9	9 46	42.9	1.36
	74*	.48	7/16	21.70	1950.1	162.5	9.48	61.2	1.68
21	60½*	.43	7/16	17.68	1235.5	117.7	8.36	43.5	1.57
20	100	.88	7/8	29.41	1655.6	165.6	7.50	52.7	1.34
	95	.81	13/16	27.94	1606.6	160.7	7.58	50.8	1.35
	90	.74	3/4	26.47	1557.6	155.8	7.67	49.0	1.36
	85	.66	5/8	25.00	1508.5	150.9	7.77	47.3	1.37
	80	.60	9/16	23.73	1466.3	146.6	7.86	45.8	1.39
	75	.65	5/8	22.06	1268.8	126.9	7.58	30.3	1.17
	70	.58	9/16	20.59	1219.8	122.0	7.70	29.0	1.19
	65	.50	1/2	19.08	1169.5	117.0	7.83	27.9	1.21
18	90	.81	13/16	26.47	1260.4	140.0	6.90	52.0	1.40
	85	.73	11/16	25.00	1220.7	135.6	6.99	50.0	1.42
	80	.64	5/8	23.53	1181.0	131.2	7.09	48.1	1.43
	75	.56	9/16	22.05	1141.3	126.8	7.19	46.2	1.45
	70	.72	11/16	20.59	921.2	102.4	6.69	24.6	1.09
	65	.64	5/8	19.12	881.5	97.9	6.79	23.5	1.11
	60	.56	9/16	17.65	841.8	93.5	6.91	22.4	1.13
	55	.46	7/16	15.93	795.6	88.4	7.07	21.2	1.15
	48*	.38	3/8	14.08	737.1	81.9	7.23	30.0	1.46
15	75	.88	7/8	22.06	691.2	92.2	5.60	30.7	1.18
	70	.78	3/4	20.59	663.7	88.5	5.68	29.0	1.19
	65	.69	11/16	19.12	636.1	84.8	5.77	27.4	1.20
	60	.59	9/16	17.67	609.0	81.2	5.87	26.0	1.21
	55	.66	5/8	16.18	511.0	68.1	5.62	17.1	1.02
	50	.56	9/16	14.71	483.4	64.5	5.73	16.0	1.04
	45	.46	7/16	13.24	455.9	60.8	5.87	15.1	1.07
	42	.41	3/8	12.48	441.8	58.9	5.95	14.6	1.08
	37½*	.33	5/16	10.91	405.5	54.1	6.10	19.9	1.35
12	55	.82	13/16	16.18	321.0	53.5	4.45	17.5	1.04
	50	.70	11/16	14.71	303.4	50.6	4.54	16.1	1.05
	45	.58	9/16	13.24	285.7	47.6	4.65	14.9	1.06
	40	.46	7/16	11.84	269.0	44.8	4.77	13.8	1.08
	35	.44	7/16	10.29	228.3	38.0	4.71	10.1	0.99
	31½	.35	5/16	9.26	215.8	36.0	4.83	9.5	1.01
	28*	.28	1/4	8.15	199.4	33.2	4.95	12.6	1.24
10	40	.75	3/4	11.76	158.7	31.7	3.67	9.5	0.90
	35	.60	9/16	10.29	146.4	29.3	3.77	8.5	0.91
	30	.46	7/16	8.82	134.2	26.8	3.90	7.7	0.93
	25	.31	5/16	7.37	122.1	24.4	4.07	6.9	0.97
	22¼*	.25	1/4	6.54	113.6	22.7	4.17	9.0	1.17
9	35	.73	11/16	10.29	111.8	24.8	3.29	7.3	0.84
	30	.57	9/16	8.82	101.9	22.6	3.40	6.4	0.85
	25	.41	3/8	7.35	91.9	20.4	3.54	5.7	0.88
	21	.29	1/4	6.31	84.9	18.9	3.67	5.2	0.90
8	25½	.54	1/2	7.50	68.4	17.1	3.02	4.8	0.80
	23	.45	7/16	6.76	64.5	16.1	3.09	4.4	0.81
	20½	.36	5/16	6.03	60.6	15.2	3.17	4.1	0.82
	18	.27	1/4	5.33	56.9	14.2	3.27	3.8	0.84
	17½*	.22	3/16	5.12	58.4	14.6	3.38	6.2	1.10
7	20	.46	7/16	5.88	42.2	12.1	2.68	3.2	0.74
	17½	.35	5/16	5.15	39.2	11.2	2.76	2.9	0.76
	15	.25	1/4	4.42	36.2	10.4	2.86	2.7	0.78
6	17¼	.48	7/16	5.07	26.2	8.7	2.27	2.4	0.68
	14¾	.35	5/16	4.34	24.0	8.0	2.35	2.1	0.69
	12¼	.23	3/16	3.61	21.8	7.3	2.46	1.9	0.72
5	14¾	.50	1/2	4.34	15.2	6.1	1.87	1.7	0.63
	12¼	.36	5/16	3.60	13.6	5.5	1.94	1.5	0.63
	9¾	.21	3/16	2.87	12.1	4.8	2.05	1.2	0.65
4	10½	.41	3/8	3.09	7.1	3.6	1.52	1.0	0.57
	9½	.34	5/16	2.79	6.8	3.4	1.55	0.9	0.58
	8½	.26	1/4	2.50	6.4	3.2	1.59	0.9	0.58
	7½	.19	3/16	2.21	6.0	3.0	1.64	0.8	0.59
3	7½	.36	3/8	2.21	2.9	1.9	1.15	0.6	0.52
	6½	.26	1/4	1.91	2.7	1.8	1.19	0.5	0.52
	5½	.17	1/8	1.63	2.5	1.7	1.23	0.5	0.53

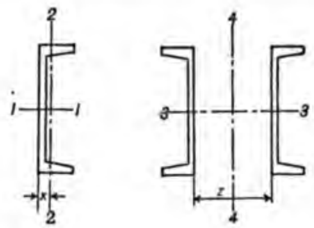
STANDARD I-BEAMS

SIZE IN.	WEIGHT LBS./FT.	WEB IN.		AREA SQ.IN.	I IN.⁴	s IN.³	r IN.	I_2 IN.⁴	r_2 IN.
24	105†	.63	5/8	30.98	2811.5	234.3	9.53	78.9	1.60
	100	.75	3/4	29.41	2379.6	198.3	8.99	48.6	1.28
	95	.69	11/16	27.94	2309.0	192.4	9.09	47.1	1.30
	90	.63	5/8	26.47	2238.4	186.5	9.20	45.7	1.31
	85	.57	9/16	25.00	2167.8	180.7	9.31	44.4	1.33
	80	.50	1/2	23.32	2087.2	173.9	9.46	42.9	1.36
20	100	.88	7/8	29.41	1655.6	165.6	7.50	52.7	1.34
	95	.81	13/16	27.94	1606.6	160.7	7.58	50.8	1.35
	90	.74	3/4	26.47	1557.5	155.8	7.67	49.0	1.36
	85	.66	5/8	25.00	1508.5	150.9	7.77	47.3	1.37
	80	.60	9/16	23.73	1466.3	146.6	7.86	45.8	1.39
	75	.65	5/8	22.06	1268.8	126.9	7.58	30.3	1.17
	70	.58	9/16	20.59	1219.8	122.0	7.70	29.0	1.19
	65	.50	1/2	19.08	1169.5	117.0	7.83	27.9	1.21
18	70	.72	11/16	20.59	921.2	102.4	6.69	24.6	1.09
	65	.64	5/8	19.12	881.5	97.9	6.79	23.5	1.11
	60	.56	9/16	17.65	841.8	93.5	6.91	22.4	1.13
	55	.46	7/16	15.93	795.6	88.4	7.07	21.2	1.15
15	100	1.18	1 3/16	29.41	900.5	120.1	5.53	51.0	1.31
	95	1.09	1 1/16	27.94	872.9	116.4	5.59	48.4	1.32
	90	.99	1	26.47	845.4	112.7	5.65	45.9	1.32
	85	.89	7/8	25.00	817.8	109.0	5.72	43.6	1.32
	80	.81	13/16	23.53	795.5	106.1	5.78	41.8	1.32
	75	.88	7/8	22.06	691.2	92.2	5.60	30.7	1.18
	70	.78	3/4	20.59	663.7	88.5	5.68	29.0	1.19
	65	.69	11/16	19.12	636.1	84.8	5.77	27.4	1.20
	60	.59	9/16	17.67	609.0	81.2	5.87	26.0	1.21
	55	.66	5/8	16.18	511.0	68.1	5.62	17.1	1.03
	50	.56	9/16	14.71	483.4	64.5	5.73	16.0	1.04
	45	.46	7/16	13.24	455.8	60.8	5.87	15.1	1.07
	42	.41	3/8	12.48	441.8	58.9	5.95	14.6	1.08
12	55	.82	13/16	16.18	321.0	53.5	4.45	17.5	1.04
	50	.70	11/16	14.71	303.4	50.6	4.54	16.1	1.05
	45	.58	9/16	13.24	285.7	47.6	4.65	14.9	1.06
	40	.46	7/16	11.84	268.9	44.8	4.77	13.8	1.08
	35	.44	7/16	10.29	228.3	38.0	4.71	10.1	0.99
	31½	.35	5/16	9.26	215.8	36.0	4.83	9.5	1.01
10	40	.75	3/4	11.76	158.7	31.7	3.67	9.5	0.90
	35	.60	9/16	10.29	146.4	29.3	3.77	8.5	0.91
	30	.46	7/16	8.82	134.2	26.8	3.90	7.7	0.93
	25	.31	5/16	7.37	122.1	24.4	4.07	6.9	0.97
9	35	.73	11/16	10.29	111.8	24.8	3.30	7.3	0.84
	30	.57	9/16	8.82	101.9	22.6	3.40	6.4	0.85
	25	.41	3/8	7.35	91.9	20 4	3.54	5.7	0.88
	21	.29	1/4	6.31	84.9	18.9	3.67	5.2	0.90
8‡	25½	.54	1/2	7.50	68.4	17.1	3.02	4.8	0.80
	23	.45	7/16	6.76	64.5	16.1	3.09	4.4	0.81
	20½	.36	5/16	6.03	60.6	15.2	3.17	4.1	0.82
	18	.27	1/4	5.33	56.9	14.2	3.27	3.8	0.84
7	20	.46	7/16	5.88	42.2	12.1	2.68	3.2	0.74
	17½	.35	5/16	5.15	39.2	11.2	2.76	2.9	0.76
	15	.25	1/4	4.42	36.2	10.4	2.86	2.7	0.78
6	17¼	.48	7/16	5.07	26.2	8.7	2.27	2.4	0.68
	14¾	.35	5/16	4.34	24.0	8.0	2.35	2.1	0.69
	12¼	.23	3/16	3.61	21.8	7.3	2.46	1.9	0.72
5	14¾	.50	1/2	4.34	15.1	6.1	1.87	1.7	0.63
	12¼	.36	5/16	3.60	13.6	5.4	1.94	1.5	0.63
	9¾	.21	3/16	2.87	12.1	4.8	2.05	1.2	0.65
4	10½	.41	3/8	3.09	7.1	3.6	1.52	1.0	0.57
	9½	.34	5/16	2.79	6.7	3.4	1.54	0.9	0.58
	8½	.26	1/4	2.50	6.4	3.2	1.59	0.9	0.58
	7½	.19	3/16	2.21	6.0	3.0	1.64	0.8	0.59
3	7½	.36	3/8	2.21	2.9	1.9	1.15	0.6	0.52
	6½	.26	1/4	1.91	2.7	1.8	1.19	0.5	0.52
	5½	.17	1/8	1.63	2.5	1.7	1.23	0.5	0.53

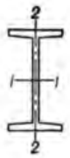
* SUPPLEMENTARY BEAM. † SPECIAL. ‡ CAMBRIA 8" I'S WEIGH 25¼, 22¾, 20¼ AND 18.

IF THE THICKNESS OF THE WEB IS MORE THAN $\frac{1}{64}$ BELOW AN EVEN SIXTEENTH, THE NEXT LOWER SIXTEENTH IS GIVEN IN THE TABLE BELOW

I = THE MOMENT OF INERTIA OF THE CROSS SECTION ABOUT THE AXIS 1-1
s = THE CORRESPONDING SECTION MODULUS ABOUT THE AXIS 1-1
r = THE CORRESPONDING RADIUS OF GYRATION ABOUT THE AXIS 1-1
I_2 = THE MOMENT OF INERTIA OF THE CROSS SECTION ABOUT THE AXIS 2-2
s_2 = THE CORRESPONDING SECTION MODULUS ABOUT THE AXIS 2-2
r_2 = THE CORRESPONDING RADIUS OF GYRATION ABOUT THE AXIS 2-2
x = THE DISTANCE FROM THE BACK OF THE WEB TO THE CENTER OF GRAVITY
z = THE DISTANCE BACK TO BACK OF WEBS WHICH MAKES $I_3 = I_4 = 2I$
* NOT STANDARD FOR ALL STEEL COMPANIES



SIZE	WEIGHT	WEB		AREA	I	s	r	I_2	s_2	r_2	x	z
IN.	LBS./FT.	IN.		SQ. IN.	IN.⁴	IN.³	IN.	IN.⁴	IN.³	IN.	IN.	IN.
15	55	.82	13/16	16.18	430.2	57.4	5.16	12.2	4.1	0.87	0.82	8.53
	50	.72	11/16	14.71	402.7	53.7	5.23	11.2	3.8	0.87	0.80	8.72
	45	.62	5/8	13.24	375.1	50.0	5.32	10.3	3.6	0.88	0.79	8.92
	40	.52	1/2	11.76	347.5	46.3	5.44	9.4	3.4	0.89	0.78	9.16
	35	.43	7/16	10.29	319.9	42.7	5.57	8.5	3.2	0.91	0.79	9.42
	33	.40	3/8	9.90	312.6	41.7	5.62	8.2	3.2	0.91	0.79	9.51
13*	50	.79	3/4	14.71	313.7	48.3	4.62	16.7	4.9	1.07	0.98	7.02
	45	.68	11/16	13.24	292.9	45.1	4.70	15.3	4.6	1.08	0.97	7.22
	40	.57	9/16	11.76	272.2	41.9	4.81	13.9	4.3	1.09	0.97	7.44
	37	.50	1/2	10.88	259.8	40.0	4.89	13.1	4.2	1.10	0.98	7.56
	35	.45	7/16	10.29	251.5	38.7	4.94	12.5	4.1	1.10	0.99	7.66
	32	.38	3/8	9.30	237.5	36.5	5.05	11.5	3.9	1.11	1.01	7.84
12	40	.76	3/4	11.76	196.9	32.8	4.09	6.6	2.5	0.75	0.72	6.60
	35	.64	5/8	10.29	179.3	29.9	4.17	5.9	2.3	0.76	0.69	6.83
	30	.51	1/2	8.82	161.7	26.9	4.28	5.2	2.1	0.77	0.68	7.06
	25	.39	3/8	7.35	144.0	24.0	4.43	4.5	1.9	0.78	0.68	7.36
	20½	.28	1/4	6.03	128.1	21.4	4.61	3.9	1.8	0.81	0.70	7.67
10	35	.82	13/16	10.29	115.5	23.1	3.35	4.7	1.9	0.67	0.70	5.17
	30	.68	11/16	8.82	103.2	20.6	3.42	4.0	1.7	0.67	0.65	5.40
	25	.53	1/2	7.35	91.0	18.2	3.52	3.4	1.5	0.68	0.62	5.66
	20	.38	3/8	5.88	78.7	15.7	3.66	2.9	1.3	0.70	0.61	5.97
	15	.24	1/4	4.46	66.9	13.4	3.87	2.3	1.2	0.72	0.64	6.33
9	25	.62	5/8	7.35	70.7	15.7	3.10	3.0	1.4	0.64	0.62	4.83
	20	.45	7/16	5.88	60.8	13.5	3.21	2.5	1.2	0.65	0.58	5.14
	15	.29	1/4	4.41	50.9	11.3	3.40	2.0	1.0	0.67	0.59	5.49
	13¼	.23	3/16	3.89	47.3	10.5	3.49	1.8	1.0	0.67	0.61	5.62
8	21¼	.58	9/16	6.25	47.8	11.9	2.76	2.3	1.1	0.60	0.59	4.22
	18¾	.49	1/2	5.51	43.8	11.0	2.82	2.0	1.0	0.60	0.57	4.37
	16¼	.40	3/8	4.78	39.9	10.0	2.89	1.8	1.0	0.61	0.56	4.53
	13¾	.31	5/16	4.04	36.0	9.0	2.98	1.6	0.9	0.62	0.56	4.72
	11¼	.22	3/16	3.35	32.3	8.1	3.10	1.3	0.8	0.63	0.58	4.92
7	19¾	.63	5/8	5.81	33.2	9.5	2.39	1.9	1.0	0.56	0.58	3.48
	17¼	.53	1/2	5.07	30.2	8.6	2.44	1.6	0.9	0.56	0.56	3.64
	14¾	.42	7/16	4.34	27.2	7.8	2.50	1.4	0.8	0.57	0.54	3.80
	12¼	.32	5/16	3.60	24.2	6.9	2.59	1.2	0.7	0.58	0.53	3.99
	9¾	.21	3/16	2.85	21.1	6.0	2.72	1.0	0.6	0.59	0.55	4.21
6	15½	.56	9/16	4.56	19.5	6.5	2.07	1.3	0.7	0.53	0.55	2.90
	13	.44	7/16	3.82	17.3	5.8	2.13	1.1	0.7	0.53	0.52	3.08
	10½	.32	5/16	3.09	15.1	5.0	2.21	0.9	0.6	0.53	0.50	3.29
	8	.20	3/16	2.38	13.0	4.3	2.34	0.7	0.5	0.54	0.52	3.51
5	11½	.48	7/16	3.38	10.4	4.2	1.75	0.8	0.5	0.49	0.51	2.35
	9	.33	5/16	2.65	8.9	3.6	1.83	0.6	0.5	0.49	0.48	2.57
	6½	.19	3/16	1.95	7.4	3.0	1.95	0.5	0.4	0.50	0.49	2.79
4	7¼	.33	5/16	2.13	4.6	2.3	1.46	0.4	0.4	0.46	0.46	1.88
	6¼	.25	1/4	1.84	4.2	2.1	1.51	0.4	0.3	0.45	0.46	1.96
	5¼	.18	3/16	1.55	3.8	1.9	1.56	0.3	0.3	0.45	0.46	2.08
3	6	.36	3/8	1.76	2.1	1.4	1.08	0.3	0.3	0.42	0.46	1.10
	5	.26	1/4	1.47	1.8	1.2	1.12	0.3	0.2	0.42	0.44	1.17
	4	.17	1/8	1.19	1.6	1.1	1.17	0.2	0.2	0.41	0.44	1.29



IF THE THICKNESS OF THE WEB IS MORE THAN $\frac{1}{64}$ BELOW AN EVEN SIXTEENTH, THE NEXT LOWER SIXTEENTH IS GIVEN IN THE TABLE BELOW

I = THE MOMENT OF INERTIA OF THE CROSS SECTION ABOUT THE AXIS 1-1
s = THE CORRESPONDING SECTION MODULUS ABOUT THE AXIS 1-1
r = THE CORRESPONDING RADIUS OF GYRATION ABOUT THE AXIS 1-1
I_2 = THE MOMENT OF INERTIA OF THE CROSS SECTION ABOUT THE AXIS 2-2
r_2 = THE CORRESPONDING RADIUS OF GYRATION ABOUT THE AXIS 2-2

BETHLEHEM I-BEAMS

SIZE	WEIGHT	WEB		AREA	I	s	r	I_2	r_2
IN.	LBS./FT.	IN.		SQ. IN.	IN.^{4}	IN.^{3}	IN.	IN.^{4}	IN.
30	120	.54	½	35.30	5239.6	349.3	12.18	165.0	2.16
28	105	.50	½	30.88	4014.1	286.7	11.40	131.5	2.06
26	90	.46	7/16	26.49	2977.2	229.0	10.60	101.2	1.95
24	84	.46	7/16	24.80	2381.9	198.5	9.80	91.1	1.92
	83	.52	½	24.59	2240.9	186.7	9.55	78.0	1.78
	73	.39	⅜	21.47	2091.0	174.3	9.87	74.4	1.86
20	82	.57	9/16	24.17	1559.8	156.0	8.03	79.9	1.82
	72	.43	7/16	21.37	1466.5	146.7	8.28	75.9	1.88
	69	.52	½	20.26	1268.9	126.9	7.91	51.2	1.59
	64	.45	7/16	18.86	1222.1	122.2	8.05	49.8	1.62
	59	.38	⅜	17.36	1172.2	117.2	8.22	48.3	1.66
18	59	.50	½	17.40	883.3	98.1	7.12	39.1	1.50
	54	.41	⅜	15.87	842.0	93.6	7.28	37.7	1.54
	52	.38	⅜	15.24	825.0	91.7	7.36	37.1	1.56
	48½	.32	5/16	14.25	798.3	88.7	7.48	36.2	1.59
15	71	.52	½	20.95	796.2	106.2	6.16	61.3	1.71
	64	.61	9/16	18.81	664.9	88.6	5.95	41.9	1.49
	54	.41	⅜	15.88	610.0	81.3	6.20	38.3	1.55
	46	.44	7/16	13.52	484.8	64.6	5.99	25.2	1.36
	41	.34	5/16	12.02	456.7	60.9	6.16	24.0	1.41
	38	.29	¼	11.27	442.6	59.0	6.27	23.4	1.44
12	36	.31	5/16	10.61	269.2	44.9	5.04	21.3	1.42
	32	.34	5/16	9.44	228.5	38.1	4.92	16.0	1.30
	28½	.25	¼	8.42	216.2	36.0	5.07	15.3	1.35
10	28½	.39	⅜	8.34	134.6	26.9	4.02	12.1	1.21
	23½	.25	¼	6.94	122.9	24.6	4.21	11.2	1.27
9	24	.37	⅜	7.04	92.1	20.5	3.62	8.8	1.12
	20	.25	¼	6.01	85.1	18.9	3.76	8.2	1.17
8	19½	.33	5/16	5.78	60.6	15.1	3.24	6.7	1.08
	17½	.25	¼	5.18	57.4	14.3	3.33	6.4	1.11

BETHLEHEM GIRDER BEAMS

SIZE	WEIGHT	WEB		AREA	I	s	r	I_2	r_2
IN.	LBS./FT.	IN.		SQ. IN.	IN.^{4}	IN.^{3}	IN.	IN.^{4}	IN
30	200	.75	¾	58.71	9150.6	610.0	12.48	630.2	3.28
	180	.69	11/16	53.00	8194.5	546.3	12.43	433.3	2.86
28	180	.69	11/16	52.86	7264.7	518.9	11.72	533.3	3.18
	165	.66	⅝	48.47	6562.7	468.8	11.64	371.9	2.77
26	160	.63	⅝	46.91	5620.8	432.4	10.95	435.7	3.05
	150	.63	⅝	43.94	5153.9	396.5	10.83	314.6	2.68
24	140	.60	9/16	41.16	4201.4	350.1	10.10	346.9	2.90
	120	.53	½	35.38	3607.3	300.6	10.10	249.4	2.66
20	140	.64	⅝	41.19	2934.7	293.5	8.44	348.9	2.91
	112	.55	9/16	32.81	2342.1	234.2	8.45	239.3	2.70
18	92	.48	7/16	27.12	1591.4	176.8	7.66	182.6	2.59
15	140	.80	13/16	41.27	1592.7	212.4	6.21	331.0	2.83
	104	.60	9/16	30.50	1220.1	162.7	6.32	213.0	2.64
	73	.43	7/16	21.49	883.4	117.8	6.41	123.2	2.39
12	70	.46	7/16	20.58	538.8	89.8	5.12	114.7	2.36
	55	.37	⅜	16.18	432.0	72.0	5.17	81.1	2.24
10	44	.31	5/16	12.95	244.2	48.8	4.34	57.3	2.10
9	38	.30	5/16	11.22	170.9	38.0	3.90	44.1	1.98
8	32½	.29	¼	9.54	114.4	28.6	3.46	32.9	1.86

EACH MEMBER WHICH IS SHIPPED SEPARATELY SHOULD BE MARKED WITH A CHARACTERISTIC LETTER OR LETTERS FOLLOWED BY A SPECIFIC NUMBER, THUS: *B 14*, *LG 2*.

ALL MEMBERS WHICH ARE INTERCHANGEABLE, EXCEPT OFFICE BUILDING COLUMNS,* SHOULD BEAR THE SAME MARKS.

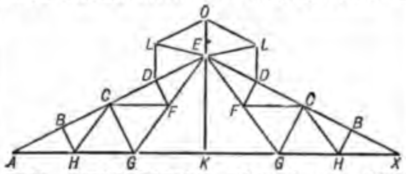
MEMBERS WHICH ARE EXACT OPPOSITES SHOULD BE MARKED RIGHT AND LEFT, THUS: *C 18*^{R}, *C 18*^{L}, THE ONE SHOWN ON THE DRAWING BEING THE RIGHT.

IN ALL OTHER CASES, MEMBERS SHOULD BEAR DIFFERENT MARKS.

THE MORE COMMON CHARACTERISTIC LETTERS USED FOR SHIPPING MARKS (ADAPTED FROM THE STANDARDS OF THE AMERICAN BRIDGE COMPANY) ARE GIVEN BELOW:—

BUILDING WORK

ANGLE BRACING	*D*
BEARING PLATES	*MP*
BUCKLED PLATES	*BP*
CAST BASES	*CB*
CASTINGS, MISCELLANEOUS, STANDARD	*A*
" " SPECIAL	*N*
COLUMNS AND POSTS, MILL BUILDINGS, ETC.	*C*
" OFFICE BUILDINGS	*
CRANE STOPS	*CS*
FLOOR PLATES	*FP*
GIRTS, AND PENT HOUSE FRAMING	*F*
GIRDERS, PLATE	*G*†
" LATTICED	*LG*
HOPPERS, BINS, AND CHUTES	*H*
I-BEAMS AND CHANNELS, MILL BUILDINGS, ETC.	*B*
" " " OFFICE "	‡
KNEE BRACES	*K*
LINTELS	*L*
MISCELLANEOUS ANGLES, PLATES, BRACKETS	*M*
PURLINS	*P*
RAFTERS	*R*
RAILINGS	*R*
RAILING POSTS	*P*
RODS, BRACING	*X*
" TIE AND SAG, LENGTH IN INS., THUS:	(48)
SIDE WALL, PARTITION, & CEILING FRAMING	*F*
SMOKE FLUES	*SF*
SPLICE PLATES	*SP*
STRUTS	*S*
TRUSSES, COMPLETE	*T*
TRUSS MEMBERS	AS FOLLOWS:—

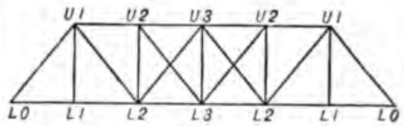


THE INTERSECTION POINTS OF ROOF TRUSSES SHOULD BE LETTERED, AND EACH SEPARATE MEMBER SHOULD BEAR THE LETTERS AT THE ENDS. THUS: *GG 2*, *EK 1*^{R}.

WHEN TRUSSES ARE SHIPPED IN SECTIONS, SUCH SECTIONS MAY BE MARKED *PT* (PART TRUSS), UNLESS SHIPPED IN HALVES, AS SHOWN IN THE FIGURE ABOVE, WHEN THE HALF *AEG* IS MARKED *AH*, AND THE HALF *XEG* IS MARKED *XH*, THE PEAK PLATE BEING SHIPPED ON THE *AH* HALF.

BRIDGE WORK

ANGLE BRACING, BETWEEN STRINGERS	*D*
" " BOTTOM LATERALS	*L*
" " TOP "	*T*
BED PLATES, EXPANSION END	*RP*
" " FIXED END	*FP*
BRACKETS	*B*
BUCKLED PLATES	*BP*
CAST PEDESTALS	*CP*
CROSS FRAMES	*CF*
END POSTS	*EP*
FLOOR BEAMS	*F*
GIRDERS, PLATE	*G*
" LATTICED	*LG*
KNEE BRACES	*K*
MISCELLANEOUS ANGLES	*M*
" PLATES	*P*
PINS	*PANEL POINT LETTER*
RAILINGS	*R*
RAILING POSTS	*P*
ROLLER NESTS	*RN*
SHOES, EXPANSION	*RS*
" FIXED	*FS*
SHOE STRUTS	*ES*
SPLICE PLATES	*SP*
STRINGERS	*S*
STRUTS, BOTTOM LATERAL	*BS*
" TOP "	*TS*
" PORTAL	*PS*
" SWAY BRACING	*SB*
TRUSSES, COMPLETE	*T*
TRUSS MEMBERS	AS FOLLOWS:—



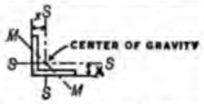
THE UPPER PANEL POINTS OF A BRIDGE TRUSS SHOULD BE MARKED *U 1*, *U 2*, ETC., AND THE LOWER PANEL POINTS *L 0*, *L 1*, ETC., SO THAT *L 1* IS UNDER *U 1*. THE END POST (*L 0–U 1*) SHOULD BE MARKED *EP*, BUT EACH OTHER MEMBER SHOULD BEAR THE MARKS OF THE PANEL POINTS BETWEEN WHICH IT EXTENDS, THUS:—*L 1-2*, *U 1-3*, *L 1-U 1*, *L 2-U 1*. THE MEMBERS OF THE LEFT HALF OF THE FAR TRUSS SHOULD BE DRAWN AND CONSIDERED RIGHT, THE OTHERS BEING MARKED *R* AND *L* ACCORDINGLY.

* Each office building column should bear either the *Numbers* of the floors between which it extends, or else the *Letters* of the floors which it supports. Thus:—Col. 13 (0-2) or *AB 13*; Col. 25 (5-6) or *EF 25*. Office building columns should be numbered consecutively on the plans so that no two bear the same mark, even though alike. The same number should be maintained from the basement to the roof, thus section *GH 16* is spliced to the top of section *EF 16*. † Each office building girder should bear the floor number in addition to the mark, thus:—*G 7* (2ND Fl.) ‡ Each office building beam should bear either the number or the letter of the floor for which it is intended, thus.—#40—1ST Fl. or *A 40*; #15—8TH Fl. or *F 15*, #30—Roof or *R 30*.

PROPERTIES OF STANDARD ANGLES

EQUAL LEGS

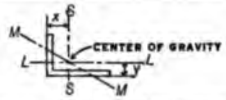
ADOPTED 1910 BY ASSOCIATION OF AMERICAN STEEL MANUFACTURERS



SIZE	THICKNESS	WEIGHT PER FT.	AREA	I_S	s_S	r_S	x	r_M
INCHES	INCHES	POUNDS	SQ. IN.	IN.^{4}	IN.^{3}	IN.	IN.	IN.
8×8	1 1/8	56.9	16.73	98.0	17.5	2.42	2.41	1.55
	1 1/16	54.0	15.87	93.5	16.7	2.43	2.39	1.56
	1	51.0	15.00	89.0	15.8	2.44	2.37	1.56
	15/16	48.1	14.12	84.3	14.9	2.44	2.34	1.56
	7/8	45.0	13.23	79.6	14.0	2.45	2.32	1.56
	13/16	42.0	12.34	74.7	13.1	2.46	2.30	1.57
	3/4	38.9	11.44	69.7	12.2	2.47	2.28	1.57
	11/16	35.8	10.53	64.6	11.2	2.48	2.25	1.58
	5/8	32.7	9.61	59.4	10.3	2.49	2.23	1.58
	9/16	29.6	8.68	54.1	9.3	2.50	2.21	1.58
	1/2	26.4	7.75	48.6	8.4	2.51	2.19	1.58
6×6	1	37.4	11.00	35.5	8.6	1.80	1.86	1.16
	15/16	35.3	10.37	33.7	8.1	1.80	1.84	1.16
	7/8	33.1	9.73	31.9	7.6	1.81	1.82	1.17
	13/16	31.0	9.09	30.1	7.2	1.82	1.80	1.17
	3/4	28.7	8.44	28.2	6.7	1.83	1.78	1.17
	11/16	26.5	7.78	26.2	6.2	1.83	1.75	1.17
	5/8	24.2	7.11	24.2	5.7	1.84	1.73	1.17
	9/16	21.9	6.43	22.1	5.1	1.85	1.71	1.18
	1/2	19.6	5.75	19.9	4.6	1.86	1.68	1.18
	7/16	17.2	5.06	17.7	4.1	1.87	1.66	1.19
	3/8	14.9	4.36	15.4	3.5	1.88	1.64	1.19
4×4	3/4	18.5	5.44	7.7	2.8	1.19	1.27	0.77
	11/16	17.1	5.03	7.2	2.6	1.19	1.25	0.77
	5/8	15.7	4.61	6.7	2.4	1.20	1.23	0.77
	9/16	14.3	4.18	6.1	2.2	1.21	1.21	0.78
	1/2	12.8	3.75	5.6	2.0	1.22	1.18	0.78
	7/16	11.3	3.31	5.0	1.8	1.23	1.16	0.78
	3/8	9.8	2.86	4.4	1.5	1.23	1.14	0.79
	5/16	8.2	2.40	3.7	1.3	1.24	1.12	0.79
3½×3½	5/8	13.6	3.98	4.3	1.8	1.04	1.10	0.68
	9/16	12.4	3.62	4.0	1.6	1.05	1.08	0.68
	1/2	11.1	3.25	3.6	1.5	1.06	1.06	0.68
	7/16	9.8	2.87	3.3	1.3	1.07	1.04	0.68
	3/8	8.5	2.48	2.9	1.2	1.07	1.01	0.69
	5/16	7.2	2.09	2.5	1.0	1.08	0.99	0.69
3×3	1/2	9.4	2.75	2.2	1.1	0.90	0.93	0.58
	7/16	8.3	2.43	2.0	0.95	0.91	0.91	0.58
	3/8	7.2	2.11	1.8	0.83	0.91	0.89	0.58
	5/16	6.1	1.78	1.5	0.71	0.92	0.87	0.59
	1/4	4.9	1.44	1.2	0.58	0.93	0.84	0.59
2½×2½	7/16	6.8	2.00	1.1	0.65	0.75	0.78	0.48
	3/8	5.9	1.73	0.98	0.57	0.75	0.76	0.48
	5/16	5.0	1.47	0.85	0.48	0.76	0.74	0.49
	1/4	4.1	1.19	0.70	0.39	0.77	0.72	0.49
	3/16	3.1	0.90	0.55	0.30	0.78	0.69	0.49
2×2	3/8	4.7	1.36	0.48	0.35	0.59	0.64	0.39
	5/16	3.9	1.15	0.42	0.30	0.60	0.61	0.39
	1/4	3.2	0.94	0.35	0.25	0.61	0.59	0.39
	3/16	2.4	0.71	0.28	0.19	0.62	0.57	0.40

UNEQUAL LEGS

ADOPTED 1910 BY ASSOCIATION OF AMERICAN STEEL MANUFACTURERS

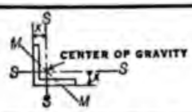


SIZE	THICKNESS	WEIGHT PER FT.	AREA	I_S	s_S	r_S	x	I_L	s_L	r_L	y	
INCHES	INCHES	POUNDS	SQ. IN.	IN.^{4}	IN.^{3}	IN.	IN.	IN.^{4}	IN.^{3}	IN.	IN.	I
6×4	7/8	27.2	7.98	27.7	7.2	1.86	2.12	9.8	3.4	1.11	1.12	0
	13/16	25.4	7.47	26.1	6.7	1.87	2.10	9.2	3.2	1.11	1.10	0
	3/4	23.6	6.94	24.5	6.2	1.88	2.08	8.7	3.0	1.12	1.08	0
	11/16	21.8	6.40	22.8	5.8	1.89	2.06	8.1	2.8	1.13	1.06	0
	5/8	20.0	5.86	21.1	5.3	1.90	2.03	7.5	2.5	1.13	1.03	0
	9/16	18.1	5.31	19.3	4.8	1.90	2.01	6.9	2.3	1.14	1.01	0
	1/2	16.2	4.75	17.4	4.3	1.91	1.99	6.3	2.1	1.15	0.99	0
	7/16	14.3	4.18	15.5	3.8	1.92	1.96	5.6	1.8	1.16	0.96	0
	3/8	12.3	3.61	13.5	3.3	1.93	1.94	4.9	1.6	1.17	0.94	0
6×3½	7/8	25.7	7.55	26.4	7.0	1.87	2.22	6.6	2.6	0.93	0.97	0
	13/16	24.0	7.06	24.9	6.6	1.88	2.20	6.2	2.4	0.94	0.95	0
	3/4	22.4	6.56	23.3	6.1	1.89	2.18	5.8	2.3	0.94	0.93	0
	11/16	20.6	6.06	21.7	5.6	1.89	2.15	5.5	2.1	0.95	0.90	0
	5/8	18.9	5.55	20.1	5.2	1.90	2.13	5.1	1.9	0.96	0.88	0
	9/16	17.1	5.03	18.4	4.7	1.91	2.11	4.7	1.8	0.96	0.86	0
	1/2	15.3	4.50	16.6	4.2	1.92	2.08	4.3	1.6	0.97	0.83	0
	7/16	13.5	3.97	14.8	3.7	1.93	2.06	3.8	1.4	0.98	0.81	0
	3/8	11.7	3.42	12.9	3.3	1.94	2.04	3.3	1.2	0.99	0.78	0
5×3½	3/4	19.8	5.81	13.9	4.3	1.55	1.75	5.6	2.2	0.98	1.00	0
	11/16	18.3	5.37	13.0	4.0	1.56	1.72	5.2	2.1	0.98	0.97	0
	5/8	16.8	4.92	12.0	3.7	1.56	1.70	4.8	1.9	0.99	0.95	0
	9/16	15.2	4.47	11.0	3.3	1.57	1.68	4.4	1.7	1.00	0.93	0
	1/2	13.6	4.00	10.0	3.0	1.58	1.66	4.0	1.6	1.01	0.91	0
	7/16	12.0	3.53	8.9	2.6	1.59	1.63	3.6	1.4	1.01	0.88	0
	3/8	10.4	3.05	7.8	2.3	1.60	1.61	3.2	1.2	1.02	0.86	0
	5/16	8.7	2.56	6.6	1.9	1.61	1.59	2.7	1.0	1.03	0.84	0
5×3	11/16	17.1	5.03	12.3	3.9	1.56	1.82	3.3	1.5	0.81	0.82	0
	5/8	15.7	4.61	11.4	3.5	1.57	1.80	3.1	1.4	0.81	0.80	0
	9/16	14.3	4.18	10.4	3.2	1.58	1.77	2.8	1.3	0.82	0.77	0
	1/2	12.8	3.75	9.5	2.9	1.59	1.75	2.6	1.1	0.83	0.75	0
	7/16	11.3	3.31	8.4	2.6	1.60	1.73	2.3	1.0	0.84	0.73	0
	3/8	9.8	2.86	7.4	2.2	1.61	1.70	2.0	0.89	0.84	0.70	0
	5/16	8.2	2.40	6.3	1.9	1.61	1.68	1.8	0.75	0.85	0.68	0
4×3	5/8	13.6	3.98	6.0	2.3	1.23	1.37	2.9	1.4	0.85	0.87	0
	9/16	12.4	3.62	5.6	2.1	1.24	1.35	2.7	1.2	0.86	0.85	0
	1/2	11.1	3.25	5.0	1.9	1.25	1.33	2.4	1.1	0.86	0.83	0
	7/16	9.8	2.87	4.5	1.7	1.25	1.30	2.2	1.0	0.87	0.80	0
	3/8	8.5	2.48	4.0	1.5	1.26	1.28	1.9	0.87	0.88	0.78	0
	5/16	7.2	2.09	3.4	1.2	1.27	1.26	1.7	0.74	0.89	0.76	0
3½×3	9/16	11.4	3.34	3.8	1.6	1.07	1.15	2.5	1.2	0.87	0.90	0
	1/2	10.2	3.00	3.5	1.5	1.07	1.13	2.3	1.1	0.88	0.88	0
	7/16	9.1	2.65	3.1	1.3	1.08	1.10	2.1	0.98	0.89	0.85	0
	3/8	7.9	2.30	2.7	1.1	1.09	1.08	1.8	0.85	0.90	0.83	0
	5/16	6.6	1.93	2.3	1.0	1.10	1.06	1.6	0.72	0.90	0.81	0
3½×2½	1/2	9.4	2.75	3.2	1.4	1.09	1.20	1.4	0.76	0.70	0.70	0
	7/16	8.3	2.43	2.9	1.3	1.09	1.18	1.2	0.68	0.71	0.68	0
	3/8	7.2	2.11	2.6	1.1	1.10	1.16	1.1	0.59	0.72	0.66	0
	5/16	6.1	1.78	2.2	0.93	1.11	1.14	0.94	0.50	0.73	0.64	0
	1/4	4.9	1.44	1.8	0.75	1.12	1.11	0.78	0.41	0.74	0.61	0
3×2½	7/16	7.6	2.21	1.9	0.93	0.92	0.98	1.2	0.66	0.73	0.73	0
	3/8	6.6	1.92	1.7	0.81	0.93	0.96	1.0	0.58	0.74	0.71	0
	5/16	5.6	1.62	1.4	0.69	0.94	0.93	0.90	0.49	0.74	0.68	0
	1/4	4.5	1.31	1.2	0.56	0.95	0.91	0.74	0.40	0.75	0.66	0
2½×2	3/8	5.3	1.55	0.91	0.55	0.77	0.83	0.51	0.36	0.58	0.58	0
	5/16	4.5	1.31	0.79	0.47	0.78	0.81	0.45	0.31	0.58	0.56	0
	1/4	3.6	1.06	0.65	0.38	0.78	0.79	0.37	0.25	0.59	.54	0
	3/16	2.8	0.81	0.51	0.29	0.79	0.76	0.29	0.20	0.60	0.51	0

EQUAL AND UNEQUAL LEGS

* NOT ROLLED BY ALL THE LEADING STEEL COMPANIES

FOR ADDITIONAL SECTIONS SEE PAGE 303

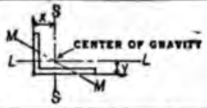


SIZE	THICKNESS	WEIGHT PER FT.	AREA	I_S	s_S	r_S	x	I_L	s_L	r_L	y	r_M
INCHES	INCHES	POUNDS	SQ. IN.	IN.⁴	IN.³	IN.	IN.	IN.⁴	IN.³	IN.	IN.	IN.
5×5	* 15/16	28.9	8.50	18.7	5.5	1.48	1.59	.....	....	....	.....	0.96
	* 7/8	27.2	7.98	17.8	5.2	1.49	1.57	.....	....	....	.....	0.96
	* 13/16	25.4	7.46	16.8	4.9	1.50	1.55	.....	....	....	.....	0.97
	* 3/4	23.6	6.94	15.7	4.5	1.50	1.52	.....	....	....	.....	0.97
	11/16	21.8	6.40	14.7	4.2	1.51	1.50	.....	....	....	.....	0.97
	5/8	20.0	5.86	13.6	3.9	1.52	1.48	.....	....	....	.....	0.97
	9/16	18.1	5.31	12.4	3.5	1.53	1.46	.....	....	....	.....	0.98
	1/2	16.2	4.75	11.3	3.2	1.54	1.43	.....	....	....	.....	0.98
	7/16	14.3	4.18	10.0	2.8	1.55	1.41	.....	....	....	.....	0.98
	3/8	12.3	3.61	8.7	2.4	1.56	1.39	.....	....	....	.....	0.99
4×4	13/16	19.9	5.84	8.1	3.0	1.18	1.29	.....	....	....	.....	0.77
	* 1/4	6.6	1.94	3.0	1.0	1.25	1.09	.....	....	....	.....	0.79
3½×3½	13/16	17.1	5.03	5.3	2.3	1.02	1.17	.....	....	....	.....	0.67
	3/4	16.0	4.69	5.0	2.1	1.03	1.15	.....	....	....	.....	0.67
	11/16	14.8	4.34	4.7	2.0	1.04	1.12	.....	....	....	.....	0.67
	* 1/4	5.8	1.69	2.0	0.79	1.09	0.97	.....	....	....	.....	0.69
3×3	* 5/8	11.5	3.36	2.6	1.3	0.88	0.98	.....	....	....	.....	0.57
	9/16	10.4	3.06	2.4	1.2	0.89	0.95	.....	....	....	.....	0.58
2½×2½	1/2	7.7	2.25	1.2	0.73	0.74	0.81	.....	....	....	.....	0.47
2¼×2¼	* 7/16	6.1	1.78	0.79	0.52	0.67	0.72	.....	....	....	.....	0.43
	* 3/8	5.3	1.55	0.70	0.45	0.67	0.70	.....	....	....	.....	0.43
	* 5/16	4.5	1.31	0.61	0.39	0.68	0.68	.....	....	....	.....	0.44
	* 1/4	3.7	1.06	0.50	0.32	0.69	0.65	.....	....	....	.....	0.44
	* 3/16	2.8	0.81	0.39	0.24	0.70	0.63	.....	....	....	.....	0.44
2×2	7/16	5.3	1.56	0.54	0.40	0.59	0.66	.....	....	....	.....	0.39
8×6	* 1	44.2	13.00	80.8	15.1	2.49	2.65	38.8	8.9	1.73	1.65	1.28
	* 15/16	41.7	12.25	76.6	14.3	2.50	2.63	36.8	8.4	1.73	1.63	1.28
	7/8	39.1	11.48	72.3	13.4	2.51	2.61	34.9	7.9	1.74	1.61	1.28
	13/16	36.5	10.72	67.9	12.5	2.52	2.59	32.8	7.4	1.75	1.59	1.29
	3/4	33.8	9.94	63.4	11.7	2.53	2.56	30.7	6.9	1.76	1.56	1.29
	11/16	31.2	9.15	58.8	10.8	2.54	2.54	28.6	6.4	1.77	1.54	1.29
	5/8	28.5	8.36	54.1	9.9	2.54	2.52	26 3	5.9	1.77	1.52	1.30
	9/16	25.7	7.56	49.3	8.9	2.55	2.50	24.0	5.3	1.78	1.50	1.30
	1/2	23.0	6.75	44.3	8.0	2.56	2.47	21.7	4.8	1.79	1.47	1.30
7×3½	* 1	32.3	9.50	45.4	10.6	2.19	2.70	7.5	3.0	0.89	0.96	0.74
	* 15/16	30.5	8.97	43.1	10.0	2.19	2.69	7.2	2.8	0.89	0.94	0.74
	7/8	28.7	8.42	40.8	9.4	2.20	2.66	6.8	2.6	0.90	0.91	0.74
	13/16	26.8	7.87	38.4	8.8	2.21	2.64	6.5	2.5	0.91	0.89	0.74
	3/4	24.9	7.31	36.0	8.2	2.22	2.62	6.1	2.3	0.91	0.87	0.74
	11/16	23.0	6.75	33.5	7.6	2.23	2.60	5.7	2.1	0.92	0.85	0.74
	5/8	21.0	6.17	30.9	7.0	2.24	2.57	5.3	2.0	0.93	0.82	0.75
	9/16	19.1	5.59	28.2	6.3	2.25	2.55	4.9	1.8	0.93	0.80	0.75
	1/2	17.0	5.00	25.4	5.7	2.25	2.53	4.4	1.6	0.94	0.78	0.75
	7/16	15.0	4.40	22.6	5.0	2.26	2.50	4.0	1.4	0.95	0.75	0.76
	* 3/8	13.0	3.80	19.6	4.3	2.27	2.48	3.5	1.3	0.96	0.73	0.76
6×4	* 1	30.6	9.00	30.8	8.0	1.85	2.17	10.8	3.8	1.09	1.17	0.85
	15/16	28.9	8.50	29.3	7.6	1.86	2.14	10.3	3.6	1.10	1.14	0.85
6×3½	* 1	28.9	8.50	29.2	7.8	1.85	2.26	7.2	2.9	0.92	1.01	0.74
	15/16	27.3	8.03	27.8	7.4	1.86	2.24	6.9	2.7	0.93	0.99	0.74

UNEQUAL LEGS

* NOT ROLLED BY ALL THE LEADING STEEL COMPANIES

FOR ADDITIONAL SECTIONS SEE PAGE 303



SIZE	THICKNESS	WEIGHT PER FT.	AREA	I_S	s_S	r_S	x	I_L	s_L	r_L	y	r_M
INCHES	INCHES	POUNDS	SQ. IN.	IN.⁴	IN.³	IN.	IN.	IN.⁴	IN.³	IN.	IN.	IN.
5×4	* 7/8	24.2	7.11	16.4	5.0	1.52	1.71	9.2	3.3	1.14	1.21	0.84
	* 13/16	22.7	6.65	15.5	4.7	1.53	1.68	8.7	3.1	1.15	1.18	0.84
	* 3/4	21.1	6.19	14.6	4.4	1.54	1.66	8.2	2.9	1.15	1.16	0.84
	11/16	19.5	5.72	13.6	4.1	1.54	1.64	7.7	2.7	1.16	1.14	0.84
	5/8	17.8	5.23	12.6	3.7	1.55	1.62	7.1	2.5	1.17	1.12	0.84
	9/16	16.2	4.75	11.6	3.4	1.56	1.60	6.6	2.3	1.18	1.10	0.85
	1/2	14.5	4.25	10.5	3.1	1.57	1.57	6.0	2.0	1.18	1.07	0.85
	7/16	12.8	3.75	9.3	2.7	1.58	1.55	5.3	1.8	1.19	1.05	0.85
	3/8	11.0	3.23	8.1	2.3	1.59	1.53	4.7	1.6	1.20	1.03	0.86
5×3½	* 7/8	22.7	6.67	15.7	4.9	1.53	1.79	6.2	2.5	0.96	1.04	0.75
	* 13/16	21.3	6.25	14.8	4.6	1.54	1.77	5.9	2.4	0.97	1.02	0.75
5×3	* 7/8	21.2	6.23	14.8	4.8	1.54	1.88	3.9	1.9	0.79	0.88	0.64
	* 13/16	19.9	5.84	14.0	4.5	1.55	1.86	3.7	1.7	0.80	0.86	0.64
	3/4	18.5	5.44	13.2	4.2	1.55	1.84	3.5	1.6	0.80	0.84	0.64
4½×3	* 13/16	18.5	5.43	10.3	3.6	1.38	1.65	3.6	1.7	0.81	0.90	0.64
	* 3/4	17.3	5.06	9.7	3.4	1.39	1.63	3.4	1.6	0.82	0.88	0.64
	* 11/16	16.0	4.68	9.1	3.1	1.39	1.60	3.2	1.5	0.83	0.85	0.64
	* 5/8	14.7	4.30	8.4	2.9	1.40	1.58	3.0	1.4	0.83	0.83	0.64
	* 9/16	13.3	3.90	7.8	2.6	1.41	1.56	2.8	1.3	0.85	0.81	0.64
	* 1/2	11.9	3.50	7.0	2.4	1.42	1.54	2.5	1.1	0.85	0.79	0.65
	* 7/16	10.6	3.09	6.3	2.1	1.43	1.51	2.3	1.0	0.85	0.76	0.65
	* 3/8	9.1	2.67	5.5	1.8	1.44	1.49	2.0	0.88	0.86	0.74	0.66
	* 5/16	7.7	2.25	4.7	1.5	1.44	1.47	1.7	0.75	0.87	0.72	0.66
4×3½	* 13/16	18.5	5.43	7.8	2.9	1.19	1.36	5.5	2.3	1.01	1.11	0.72
	* 3/4	17.3	5.06	7.3	2.8	1.20	1.34	5.2	2.1	1.01	1.09	0.72
	* 11/16	16.0	4.68	6.9	2.6	1.21	1.32	4.9	2.0	1.02	1.07	0.72
	5/8	14.7	4.30	6.4	2.4	1.22	1.29	4.5	1.8	1.03	1.04	0.72
	9/16	13.3	3.90	5.9	2.1	1.23	1.27	4.2	1.7	1.03	1.02	0.72
	1/2	11.9	3.50	5.3	1.9	1.23	1.25	3.8	1.5	1.04	1.00	0.72
	7/16	10.6	3.09	4.8	1.7	1.24	1.23	3.4	1.3	1.05	0.98	0.72
	3/8	9.1	2.67	4.2	1.5	1.25	1.21	3.0	1.2	1.06	0.96	0.73
	5/16	7.7	2.25	3.6	1.3	1.26	1.18	2.6	1.0	1.07	0.93	0.73
4×3	13/16	17.1	5.03	7.3	2.9	1.21	1.44	3.5	1.7	0.83	0.94	0.64
	3/4	16.0	4.69	6.9	2.7	1.22	1.42	3.3	1.6	0.84	0.92	0.64
	11/16	14.8	4.34	6.5	2.5	1.22	1.39	3.1	1.5	0.84	0.89	0.64
3½×3	* 13/16	15.8	4.62	5.0	2.2	1.04	1.23	3.3	1.7	0.85	0.98	0.62
	* 3/4	14.7	4.31	4.7	2.1	1.04	1.21	3.1	1.5	0.85	0.96	0.62
	* 11/16	13.6	4.00	4.4	1.9	1.05	1.19	3.0	1.4	0.86	0.94	0.62
	5/8	12.5	3.67	4.1	1.8	1.06	1.17	2.8	1.3	0.87	0.92	0.62
	* 1/4	5.4	1.56	1.9	0.78	1.11	1.04	0.91	0.58	0.91	0.79	0.63
3½×3½	* 11/16	12.5	3.65	4.1	1.9	1.06	1.27	1.7	0.99	0.69	0.77	0.53
	5/8	11.5	3.36	3.8	1.7	1.07	1.25	1.6	0.92	0.69	0.75	0.53
	9/16	10.4	3.06	3.6	1.6	1.08	1.23	1.5	0.84	0.70	0.73	0.53
3×2½	9/16	9.5	2.78	2.3	1.2	0.91	1.02	1.4	0.82	0.72	0.77	0.52
	1/2	8.5	2.50	2.1	1.0	0.91	1.00	1.3	0.74	0.72	0.75	0.52
3×2	1/2	7.7	2.25	1.9	1.0	0.92	1.08	0.67	0.47	0.55	0.58	0.43
	7/16	6.8	2.00	1.7	0.89	0.93	1.06	0.61	0.42	0.55	0.56	0.43
	3/8	5.9	1.73	1.5	0.78	0.94	1.04	0.54	0.37	0.56	0.54	0.43
	5/16	5.0	1.47	1.3	0.66	0.95	1.02	0.47	0.32	0.57	0.52	0.43
	1/4	4.1	1.19	1.1	0.54	0.95	0.99	0.39	0.25	0.57	0.49	0.43
	* 3/16	3.1	0.90	0.8	0.41	0.97	0.97	0.31	0.20	0.58	0.47	0.44
2½×2	* 1/2	6.8	2.00	1.1	0.70	0.75	0.88	0.64	0.46	0.56	0.63	0.42
	* 7/16	6.1	1.78	1.0	0.62	0.76	0.85	0.58	0.41	0.57	0.60	0.42

NET AREAS IN SQ. INS., AND ALLOWABLE TENSILE STRENGTH IN THOUSANDS OF LBS. AT 16,000 LBS. PER SQ. IN., FOR TWO ANGLES

EQUAL LEGS

DIAM. OF HOLES 1/8" LARGER THAN DIAM. OF RIVETS

SIZE	THICK-NESS	GROSS AREA	ONE HOLE IN EACH ANGLE DEDUCTED								TWO HOLES IN EACH ANGLE DEDUCTED							
DIAM. OF RIVETS			5/8		3/4		7/8		1		5/8		3/4		7/8		1	
			NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS
8×8	1 1/8	33.46	...	...	...	...	31.20	499.2	30.92	494.7	...	...	...	...	28.94	463.0	28.38	454.1
	1 1/16	31.74	...	...	...	...	29.62	473.9	29.34	469.4	...	...	...	...	27.50	440.0	26.94	431.0
	1	30.00	...	...	28.24	451.8	28.00	448.0	27.74	443.8	...	...	26.48	423.7	26.00	416.0	25.48	407.7
	15/16	28.24	...	...	26.60	425.6	26.36	421.8	26.14	418.2	...	...	24.96	399.4	24.48	391.7	24.04	384.6
	7/8	26.46	...	...	24.92	398.7	24.70	395.2	24.50	392.0	...	...	23.38	374.1	22.94	367.0	22.54	360.6
	13/16	24.68	...	...	23.26	372.2	23.06	369.0	22.86	365.8	...	...	21.84	349.4	21.44	343.0	21.04	336.6
	3/4	22.88	21.76	348.2	21.56	345.0	21.38	342.1	21.20	339.2	20.64	330.2	20.24	323.8	19.88	318.1	19.52	312.2
	11/16	21.06	20.02	320.3	19.86	317.8	19.68	314.9	19.52	312.3	18.98	303.7	18.66	298.6	18.30	292.8	17.98	287.7
	5/8	19.22	18.28	292.5	18.12	289.9	17.96	287.4	17.82	285.1	17.34	277.4	17.02	272.3	16.70	267.2	16.42	262.7
	9/16	17.36	16.52	264.3	16.38	262.1	16.24	259.8	16.10	257.6	15.68	250.9	15.40	246.4	15.12	241.9	14.84	237.4
	1/2	15.50	14.74	235.8	14.62	233.9	14.50	232.0	14.38	230.1	13.98	223.7	13.74	219.8	13.50	216.0	13.26	212.2
6×6	1	22.00	...	...	20.24	323.8	20.00	320.0	19.74	315.8	...	...	18.48	295.7	18.00	288.0	17.48	279.7
	15/16	20.74	...	...	19.10	305.6	18.86	301.8	18.64	298.2	...	...	17.46	279.4	16.98	271.7	16.54	264.6
	7/8	19.46	...	...	17.92	286.7	17.70	283.2	17.50	280.0	...	...	16.38	262.1	15.94	255.0	15.54	248.6
	13/16	18.18	...	...	16.76	268.2	16.56	265.0	16.36	261.8	...	...	15.34	245.4	14.94	239.0	14.54	232.6
	3/4	16.88	15.76	252.2	15.56	249.0	15.38	246.1	15.20	243.2	14.64	234.2	14.24	227.8	13.88	222.1	13.52	216.3
	11/16	15.56	14.52	232.3	14.36	229.8	14.18	226.9	14.02	224.3	13.48	215.7	13.16	210.6	12.80	204.8	12.48	199.7
	5/8	14.22	13.28	212.5	13.12	209.9	12.96	207.4	12.82	205.1	12.34	197.4	12.02	192.3	11.70	187.2	11.42	182.7
	9/16	12.86	12.02	192.3	11.88	190.1	11.74	187.8	11.60	185.6	11.18	178.9	10.90	174.4	10.62	169.9	10.34	165.4
	1/2	11.50	10.74	171.8	10.62	169.9	10.50	168.0	10.38	166.1	9.98	159.7	9.74	155.8	9.50	152.0	9.26	148.2
	7/16	10.12	9.46	151.4	9.36	149.8	9.24	147.8	9.14	146.2	8.80	140.8	8.60	137.6	8.36	133.8	8.16	130.6
	3/8	8.72	8.16	130.6	8.06	129.0	7.96	127.4	7.88	126.1	7.60	121.6	7.40	118.4	7.20	115.2	7.04	112.6
4×4	3/4	10.88	9.76	156.2	9.56	153.0	9.38	150.1	9.20	147.2	8.64	138.2	8.24	131.8	7.88	126.1	7.52	120.2
	11/16	10.06	9.02	144.3	8.86	141.8	8.68	138.9	8.52	136.3	7.98	127.7	7.66	122.6	7.30	116.8	6.98	111.7
	5/8	9.22	8.28	132.5	8.12	129.9	7.96	127.4	7.82	125.1	7.34	117.4	7.02	112.3	6.70	107.2	6.42	102.7
	9/16	8.36	7.52	120.3	7.38	118.1	7.24	115.8	7.10	113.6	6.68	106.9	6.40	102.4	6.12	97.9	5.84	93.4
	1/2	7.50	6.74	107.8	6.62	105.9	6.50	104.0	6.38	102.1	5.98	95.7	5.74	91.8	5.50	88.0	5.26	84.2
	7/16	6.62	5.96	95.4	5.86	93.8	5.74	91.8	5.64	90.2	5.30	84.8	5.10	81.6	4.86	77.8	4.66	74.6
	3/8	5.72	5.16	82.6	5.06	81.0	4.96	79.4	4.88	78.1	4.60	73.6	4.40	70.1	4.20	67.2	4.04	64.6
	5/16	4.80	4.34	69.4	4.26	68.2	4.18	66.9	4.10	65.6	3.88	62.1	3.72	59.5	3.56	57.0	3.40	54.4
3½×3½	5/8	7.96	7.02	112.3	6.86	109.8	6.70	107.2	...	...	6.08	97.3	5.76	92.2	5.44	87.0	...	...
	9/16	7.24	6.40	102.4	6.26	100.2	6.12	97.9	...	...	5.56	89.0	5.28	84.5	5.00	80.0	...	...
	1/2	6.50	5.74	91.8	5.62	89.9	5.50	88.0	...	...	4.98	79.7	4.74	75.8	4.50	72.0	...	...
	7/16	5.74	5.08	81.3	4.98	79.7	4.86	77.8	...	...	4.42	70.7	4.22	67.5	3.98	63.7	...	...
	3/8	4.96	4.40	70.4	4.30	68.8	4.20	67.2	...	...	3.84	61.4	3.64	58.2	3.44	55.0	...	...
	5/16	4.18	3.72	59.5	3.64	58.2	3.56	57.0	...	...	3.26	52.2	3.10	49.6	2.94	47.0	...	...
3×3	1/2	5.50	4.74	75.8	4.62	73.9	4.50	72.0	...	...	3.98	63.7	3.74	59.8	...	...	...	...
	7/16	4.86	4.20	67.2	4.10	65.6	3.98	63.7	...	...	3.54	56.6	3.34	53.4	...	...	...	...
	3/8	4.22	3.66	58.6	3.56	57.0	3.46	55.4	...	...	3.10	49.6	2.90	46.4	...	...	...	...
	5/16	3.56	3.10	49.6	3.02	48.3	2.94	47.0	...	...	2.64	42.2	2.48	39.7	...	...	...	...
	1/4	2.88	2.50	40.0	2.44	39.0	2.38	38.1	...	...	2.12	33.9	2.00	32.0	...	...	...	...
2½×2½	7/16	4.00	3.34	53.4	3.24	51.8	...	...	...	...	2.68	42.9	...	...	...	...	...	...
	3/8	3.46	2.90	46.4	2.80	44.8	...	...	...	...	2.34	37.4	...	...	...	...	...	...
	5/16	2.94	2.48	39.7	2.40	38.4	...	...	...	...	2.02	32.3	...	...	...	...	...	...
	1/4	2.38	2.00	32.0	1.94	31.0	...	...	...	...	1.62	25.9	...	...	...	...	...	...
	3/16	1.80	1.52	24.3	1.48	23.7	...	...	...	...	1.24	19.8	...	...	...	...	...	...
2×2	3/8	2.72	2.16	34.6	...	...	...	...	...	...	...	...	...	...	...	...	...	...
	5/16	2.30	1.84	29.4	...	...	...	...	...	...	...	...	...	...	...	...	...	...
	1/4	1.88	1.50	24.0	...	...	...	...	...	...	...	...	...	...	...	...	...	...
	3/16	1.42	1.14	18.2	...	...	...	...	...	...	...	...	...	...	...	...	...	...

UNEQUAL LEGS

DIAM. OF HOLES 1/8" LARGER THAN DIAM. OF RIVETS

SIZE	THICK-NESS	GROSS AREA	ONE HOLE IN EACH ANGLE DEDUCTED								TWO HOLES IN EACH ANGLE DEDUCTED							
DIAM. OF RIVETS			5/8		3/4		7/8		1		5/8		3/4		7/8		1	
			NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS	NET AREA	STRESS
6×4	7/8	15.96	...	...	14.42	230.7	14.20	227.2	14.00	224.0	...	...	12.88	206.1	12.44	199.0	12.04	192.6
	13/16	14.94	...	...	13.52	216.3	13.32	213.1	13.12	209.9	...	...	12.10	193.6	11.70	187.2	11.30	180.8
	3/4	13.88	12.76	204.2	12.56	201.0	12.38	198.1	12.20	195.2	11.64	186.2	11.24	179.8	10.88	174.1	10.52	168.3
	11/16	12.80	11.76	188.2	11.60	185.6	11.42	182.7	11.26	180.2	10.72	171.5	10.40	166.4	10.04	160.6	9.72	155.5
	5/8	11.72	10.78	172.5	10.62	169.9	10.46	167.4	10.32	165.1	9.84	157.4	9.52	152.3	9.20	147.2	8.92	142.7
	9/16	10.62	9.78	156.5	9.64	154.2	9.50	152.0	9.36	149.8	8.94	143.0	8.66	138.6	8.38	134.1	8.10	129.6
	1/2	9.50	8.74	139.8	8.62	137.9	8.50	136.0	8.38	134.1	7.98	127.7	7.74	123.8	7.50	120.0	7.26	116.2
	7/16	8.36	7.70	123.2	7.60	121.6	7.48	119.7	7.38	118.1	7.04	112.6	6.84	109.4	6.60	105.6	6.40	102.4
	3/8	7.22	6.66	106.6	6.56	105.0	6.46	103.4	6.38	102.1	6.10	97.6	5.90	94.4	5.70	91.2	5.54	88.6
6×3½	7/8	15.10	...	...	13.56	217.0	13.34	213.4	13.14	210.2	...	...	12.02	192.3	11.58	185.3	...	...
	13/16	14.12	...	...	12.70	203.2	12.50	200.0	12.30	196.8	...	...	11.28	180.5	10.88	174.1	...	...
	3/4	13.12	12.00	192.0	11.80	188.8	11.62	185.9	11.44	183.0	10.88	174.1	10.48	167.7	10.12	161.9	...	...
	11/16	12.12	11.08	177.3	10.92	174.7	10.74	171.8	10.58	169.3	10.04	160.6	9.72	155.5	9.36	149.8	...	...
	5/8	11.10	10.16	162.6	10.00	160.0	9.84	157.4	9.70	155.2	9.22	147.5	8.90	142.4	8.58	137.3	...	...
	9/16	10.06	9.22	147.5	9.08	145.3	8.94	143.0	8.80	140.8	8.38	134.1	8.10	129.6	7.82	125.1	...	...
	1/2	9.00	8.24	131.8	8.12	129.9	8.00	128.0	7.88	126.1	7.48	119.7	7.24	115.8	7.00	112.0	...	...
	7/16	7.94	7.28	116.5	7.18	114.9	7.06	113.0	6.96	111.4	6.62	105.9	6.42	102.7	6.18	98.9	...	...
	3/8	6.84	6.28	100.5	6.18	98.9	6.08	97.3	6.00	96.0	5.72	91.5	5.52	88.3	5.32	85.1	...	...
5×3½	3/4	11.62	10.50	168.0	10.30	164.8	10.12	161.9	9.94	159.0	9.38	150.1	8.98	143.7	8.62	137.9	...	...
	11/16	10.74	9.70	155.2	9.54	152.6	9.36	149.8	9.20	147.2	8.66	138.6	8.34	133.4	7.98	127.7	...	...
	5/8	9.84	8.90	142.4	8.74	139.8	8.58	137.3	8.44	135.0	7.96	127.4	7.64	122.2	7.32	117.1	...	...
	9/16	8.94	8.10	129.6	7.96	127.4	7.82	125.1	7.68	122.9	7.26	116.2	6.98	111.7	6.70	107.2	...	...
	1/2	8.00	7.24	115.8	7.12	113.9	7.00	112.0	6.88	110.1	6.48	103.7	6.24	99.8	6.00	96.0	...	...
	7/16	7.06	6.40	102.4	6.30	100.8	6.18	98.9	6.08	97.3	5.74	91.8	5.54	88.6	5.30	84.8	...	...
	3/8	6.10	5.54	88.6	5.44	87.0	5.34	85.4	5.26	84.2	4.98	79.7	4.78	76.5	4.58	73.3	...	...
	5/16	5.12	4.66	74.6	4.58	73.3	4.50	72.0	4.42	70.7	4.20	67.2	4.04	64.6	3.88	62.1	...	...
5×3	11/16	10.06	9.02	144.3	8.86	141.8	8.68	138.9	8.52	136.3	7.98	127.7	7.66	122.6	7.30	116.8	...	...
	5/8	9.22	8.28	132.5	8.12	129.9	7.96	127.4	7.82	125.1	7.34	117.4	7.02	112.3	6.70	107.2	...	...
	9/16	8.36	7.52	120.3	7.38	118.1	7.24	115.8	7.10	113.6	6.68	106.9	6.40	102.4	6.12	97.9	...	...
	1/2	7.50	6.74	107.8	6.62	105.9	6.50	104.0	6.38	102.1	5.98	95.7	5.74	91.8	5.50	88.0	...	...
	7/16	6.62	5.96	95.4	5.86	93.8	5.74	91.8	5.64	90.2	5.30	84.8	5.10	81.6	4.86	77.8	...	...
	3/8	5.72	5.16	82.6	5.06	81.0	4.96	79.4	4.88	78.1	4.60	73.6	4.40	70.4	4.20	67.2	...	...
	5/16	4.80	4.34	69.4	4.26	68.2	4.18	66.9	4.10	65.6	3.88	62.1	3.72	59.5	3.56	57.0	...	...
4×3	5/8	7.96	7.02	112.3	6.86	109.8	6.70	107.2	6.56	105.0	6.08	97.3	5.76	92.2	5.44	87.0	...	...
	9/16	7.24	6.40	102.4	6.26	100.2	6.12	97.9	5.98	95.7	5.56	89.0	5.28	84.5	5.00	80.0	...	...
	1/2	6.50	5.74	91.8	5.62	89.9	5.50	88.0	5.38	86.1	4.98	79.7	4.74	75.8	4.50	72.0	...	...
	7/16	5.74	5.08	81.3	4.98	79.7	4.86	77.8	4.76	76.2	4.42	70.7	4.22	67.5	3.98	63.7	...	...
	3/8	4.96	4.40	70.4	4.30	68.8	4.20	67.2	4.12	65.9	3.84	61.4	3.64	58.2	3.44	55.0	...	...
	5/16	4.18	3.72	59.5	3.64	58.2	3.56	57.0	3.48	55.7	3.26	52.2	3.10	49.6	2.94	47.0	...	...
3½×3	9/16	6.68	5.84	93.4	5.70	91.2	5.56	80.0	...	...	5.00	80.0	4.72	75.5	4.44	71.0	...	...
	1/2	6.00	5.24	83.8	5.12	81.9	5.00	80.0	...	...	4.48	71.7	4.24	67.8	4.00	64.0	...	...
	7/16	5.30	4.64	74.2	4.54	72.6	4.42	70.7	...	...	3.98	63.7	3.78	60.5	3.54	56.6	...	...
	3/8	4.60	4.04	64.6	3.94	63.0	3.84	61.4	...	...	3.48	55.7	3.28	52.5	3.08	49.3	...	...
	5/16	3.86	3.40	54.4	3.32	53.1	3.24	51.8	...	...	2.94	47.0	2.78	44.5	2.62	41.9	...	...
3½×2½	1/2	5.50	4.74	75.8	4.62	73.9	4.50	72.0	...	...	3.98	63.7	3.74	59.8	...	...	...	...
	7/16	4.86	4.20	67.2	4.10	65.6	3.98	63.7	...	...	3.54	56.6	3.34	53.4	...	...	...	...
	3/8	4.22	3.66	58.6	3.56	57.0	3.46	55.4	...	...	3.10	49.6	2.90	46.4	...	...	...	...
	5/16	3.56	3.10	49.6	3.02	48.3	2.94	47.0	...	...	2.64	42.2	2.48	39.7	...	...	...	...
	1/4	2.88	2.50	40.0	2.44	39.0	2.38	38.1	...	...	2.12	33.9	2.00	32.0	...	...	...	...
3×2½	7/16	4.42	3.76	60.2	3.66	58.6	3.56	57.0	...	...	3.10	49.6	...	...	...	...	...	...
	3/8	3.84	3.28	52.5	3.18	50.9	3.08	49.3	...	...	2.72	43.5	...	...	...	...	...	...
	5/16	3.24	2.78	44.5	2.70	43.2	2.62	41.9	...	...	2.32	37.1	...	...	...	...	...	...
	1/4	2.62	2.24	35.8	2.18	34.9	2.12	33.9	...	...	1.86	29.8	...	...	...	...	...	...
2½×2	3/8	3.10	2.54	40.6	2.44	39.0	...	...	...	...	...	...	...	...	...	...	...	...
	5/16	2.62	2.16	34.6	2.08	33.3	...	...	...	...	...	...	...	...	...	...	...	...
	1/4	2.12	1.74	27.8	1.68	26.9	...	...	...	...	...	...	...	...	...	...	...	...
	3/16	1.62	1.34	21.4	1.30	20.8	...	...	...	...	...	...	...	...	...	...	...	...

l=LENGTH OF MEMBER IN INCHES
r=LEAST RADIUS OF GYRATION IN INCHES
VALUES BELOW DASHED LINES EXCEED 14,000

UNIT STRESSES IN THOUSANDS OF LBS. PER SQ. IN. AT 16,000−70 l/r FOR DIFFERENT VALUES OF l AND r

FOR VALUES BELOW HEAVY FULL LINES THE RATIO l/r DOES NOT EXCEED 100
" " BETWEEN " " " " " " " " 120
" " ABOVE " " " " " " " " 150

LENGTH OF MEMBER IN FEET

r	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
0.3	7.6																	
0.4	9.7	7.6	5.5															
0.5	11.0	9.3	7.6	5.9														
0.6	11.8	10.4	9.0	7.6	6.2													
0.7	12.4	11.2	10.0	8.8	7.6	6.4												
0.8	12.8	11.8	10.7	9.7	8.6	7.6	6.6	5.5										
0.9	13.2	12.3	11.3	10.4	9.5	8.5	7.6	6.7	5.7									
1.0	13.5	12.6	11.8	11.0	10.1	9.3	8.4	7.6	6.8	5.9								
1.1	13.7	12.9	12.2	11.4	10.6	9.9	9.1	8.4	7.6	6.8	6.1							
1.2	13.9	13.2	12.5	11.8	11.1	10.4	9.7	9.0	8.3	7.6	6.9	6.2	5.5					
1.3	14.1	13.4	12.8	12.1	11.5	10.8	10.2	9.5	8.9	8.2	7.6	6.9	6.3	5.7				
1.4	14.2	13.6	13.0	12.4	11.8	11.2	10.6	10.0	9.4	8.8	8.2	7.6	7.0	6.4	5.8			
1.5	14.3	13.8	13.2	12.6	12.1	11.5	11.0	10.4	9.8	9.3	8.7	8.2	7.6	7.0	6.5	5.9		
1.6	14.4	13.9	13.4	12.8	12.3	11.8	11.3	10.7	10.2	9.7	9.2	8.6	8.1	7.6	7.1	6.6	6.0	5.5
1.7	14.5	14.0	13.5	13.0	12.5	12.0	11.5	11.1	10.6	10.1	9.6	9.1	8.6	8.1	7.6	7.1	6.6	6.1
1.8	14.6	14.1	13.7	13.2	12.7	12.3	11.8	11.3	10.9	10.4	9.9	9.5	9.0	8.5	8.1	7.6	7.1	6.7
1.9	14.7	14.2	13.8	13.3	12.9	12.5	12.0	11.6	11.1	10.7	10.2	9.8	9.4	8.9	8.5	8.0	7.6	7.2
2.0	14.7	14.3	13.9	13.5	13.1	12.6	12.2	11.8	11.4	11.0	10.5	10.1	9.7	9.3	8.9	8.4	8.0	7.6
2.1	14.8	14.4	14.0	13.6	13.2	12.8	12.4	12.0	11.6	11.2	10.8	10.4	10.0	9.6	9.2	8.8	8.4	8.0
2.2	14.9	14.5	14.1	13.7	13.3	12.9	12.6	12.2	11.8	11.4	11.0	10.7	10.3	9.9	9.5	9.1	8.7	8.4
2.3	14.9	14.5	14.2	13.8	13.4	13.1	12.7	12.3	12.0	11.6	11.2	10.9	10.5	10.2	9.8	9.4	9.1	8.7
2.4	14.9	14.6	14.2	13.9	13.5	13.2	12.8	12.5	12.1	11.8	11.4	11.1	10.7	10.4	10.0	9.7	9.3	9.0
2.5	15.0	14.7	14.3	14.0	13.6	13.3	13.0	12.6	12.3	12.0	11.6	11.3	11.0	10.6	10.3	9.9	9.6	9.3
2.6	15.0	14.7	14.4	14.1	13.7	13.4	13.1	12.8	12.4	12.1	11.8	11.5	11.1	10.8	10.5	10.2	9.9	9.5
2.7	15.1	14.8	14.4	14.1	13.8	13.5	13.2	12.9	12.6	12.3	12.0	11.6	11.3	11.0	10.7	10.4	10.1	9.8
2.8	15.1	14.8	14.5	14.2	13.9	13.6	13.3	13.0	12.7	12.4	12.1	11.8	11.5	11.2	10.9	10.6	10.3	10.0
2.9	15.1	14.8	14.5	14.3	14.0	13.7	13.4	13.1	12.8	12.5	12.2	11.9	11.7	11.4	11.1	10.8	10.5	10.2
3.0	15.2	14.9	14.6	14.3	14.0	13.8	13.5	13.2	12.9	12.6	12.4	12.1	11.8	11.5	11.2	11.0	10.7	10.4
3.1	15.2	14.9	14.6	14.4	14.1	13.8	13.6	13.3	13.0	12.7	12.5	12.2	11.9	11.7	11.4	11.1	10.8	10.6
3.2	15.2	14.9	14.7	14.4	14.2	13.9	13.6	13.4	13.1	12.8	12.6	12.3	12.1	11.8	11.5	11.3	11.0	10.7
3.3	15.2	15.0	14.7	14.5	14.2	14.0	13.7	13.5	13.2	12.9	12.7	12.4	12.2	11.9	11.7	11.4	11.2	10.9
3.4	15.3	15.0	14.8	14.5	14.3	14.0	13.8	13.5	13.3	13.0	12.8	12.5	12.3	12.0	11.8	11.5	11.3	11.1
3.5	15.3	15.0	14.8	14.6	14.3	14.1	13.8	13.6	13.4	13.1	12.9	12.6	12.4	12.2	11.9	11.7	11.4	11.2
3.6	15.3	15.1	14.8	14.6	14.4	14.1	13.9	13.7	13.4	13.2	13.0	12.7	12.5	12.3	12.0	11.8	11.6	11.3
3.7	15.3	15.1	14.9	14.6	14.4	14.2	14.0	13.7	13.5	13.3	13.0	12.8	12.6	12.4	12.1	11.9	11.7	11.5
3.8	15.3	15.1	14.9	14.7	14.5	14.2	14.0	13.8	13.6	13.3	13.1	12.9	12.7	12.5	12.2	12.0	11.8	11.6
3.9	15.4	15.1	14.9	14.7	14.5	14.3	14.1	13.8	13.6	13.4	13.2	13.0	12.8	12.5	12.3	12.1	11.9	11.7
4.0	15.4	15.2	14.9	14.7	14.5	14.3	14.1	13.9	13.7	13.5	13.3	13.1	12.8	12.6	12.4	12.2	12.0	11.8
4.1	15.4	15.2	15.0	14.8	14.6	14.4	14.2	14.0	13.7	13.5	13.3	13.1	12.9	12.7	12.5	12.3	12.1	11.9
4.2	15.4	15.2	15.0	14.8	14.6	14.4	14.2	14.0	13.8	13.6	13.4	13.2	13.0	12.8	12.6	12.4	12.2	12.0
4.3	15.4	15.2	15.0	14.8	14.6	14.4	14.2	14.0	13.9	13.7	13.5	13.3	13.1	12.9	12.7	12.5	12.3	12.1
4.4	15.4	15.2	15.0	14.9	14.7	14.5	14.3	14.1	13.9	13.7	13.5	13.3	13.1	12.9	12.8	12.6	12.4	12.2
4.5	15.4	15.3	15.1	14.9	14.7	14.5	14.3	14.1	13.9	13.8	13.6	13.4	13.2	13.0	12.8	12.6	12.4	12.3
4.6	15.5	15.3	15.1	14.9	14.7	14.5	14.4	14.2	14.0	13.8	13.6	13.4	13.3	13.1	12.9	12.7	12.5	12.3
4.7	15.5	15.3	15.1	14.9	14.7	14.6	14.4	14.2	14.0	13.9	13.7	13.5	13.3	13.1	13.0	12.8	12.6	12.4
4.8	15.5	15.3	15.1	14.9	14.8	14.6	14.4	14.2	14.1	13.9	13.7	13.5	13.4	13.2	13.0	12.8	12.7	12.5
4.9	15.5	15.3	15.1	15.0	14.8	14.6	14.5	14.3	14.1	13.9	13.8	13.6	13.4	13.3	13.1	12.9	12.7	12.6
5.0	15.5	15.3	15.2	15.0	14.8	14.7	14.5	14.3	14.1	14.0	13.8	13.6	13.5	13.3	13.1	13.0	12.8	12.6
5.1	15.5	15.3	15.2	15.0	14.8	14.7	14.5	14.4	14.2	14.0	13.9	13.7	13.5	13.4	13.2	13.0	12.9	12.7
5.2	15.5	15.4	15.2	15.0	14.9	14.7	14.5	14.4	14.2	14.1	13.9	13.7	13.6	13.4	13.3	13.1	12.9	12.8
5.3	15.5	15.4	15.2	15.0	14.9	14.7	14.6	14.4	14.3	14.1	13.9	13.8	13.6	13.5	13.3	13.1	13.0	12.8
5.4	15.5	15.4	15.2	15.1	14.9	14.8	14.6	14.4	14.3	14.1	14.0	13.8	13.7	13.5	13.4	13.2	13.0	12.9
5.5	15.5	15.4	15.2	15.1	14.9	14.8	14.6	14.5	14.3	14.2	14.0	13.9	13.7	13.6	13.4	13.2	13.1	12.9
5.6	15.5	15.4	15.2	15.1	14.9	14.8	14.6	14.5	14.3	14.2	14.0	13.9	13.7	13.6	13.4	13.3	13.1	13.0
5.7	15.6	15.4	15.3	15.1	15.0	14.8	14.7	14.5	14.4	14.2	14.1	13.9	13.8	13.6	13.5	13.3	13.2	13.0
5.8	15.6	15.4	15.3	15.1	15.0	14.8	14.7	14.5	14.4	14.3	14.1	14.0	13.8	13.7	13.5	13.4	13.2	13.1
5.9	15.6	15.4	15.3	15.1	15.0	14.9	14.7	14.6	14.4	14.3	14.1	14.0	13.9	13.7	13.6	13.4	13.3	13.1
6.0	15.6	15.4	15.3	15.2	15.0	14.9	14.7	14.6	14.5	14.3	14.2	14.0	13.9	13.8	13.6	13.5	13.3	13.2

r	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
0.3																				
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1.6																				
1.7	5.6																			
1.8	6.2	5.7																		
1.9	6.7	6.3	5.8																	
2.0	7.2	6.8	6.3	5.9	5.5															
2.1	7.6	7.2	6.8	6.4	6.0	5.6														
2.2	8.0	7.6	7.2	6.8	6.4	6.1	5.7													
2.3	8.3	8.0	7.6	7.2	6.9	6.5	6.1	5.8												
2.4	8.6	8.3	7.9	7.6	7.2	6.9	6.6	6.2	5.8	5.5										
2.5	8.9	8.6	8.3	7.9	7.6	7.3	6.9	6.6	6.3	5.9	5.6									
2.6	9.2	8.9	8.6	8.2	7.9	7.6	7.3	6.9	6.6	6.3	6.0	5.7								
2.7	9.5	9.2	8.8	8.5	8.2	7.9	7.6	7.3	7.0	6.7	6.4	6.0	5.7							
2.8	9.7	9.4	9.1	8.8	8.5	8.2	7.9	7.6	7.3	7.0	6.7	6.4	6.1	5.8	5.5					
2.9	9.9	9.6	9.3	9.0	8.8	8.5	8.2	7.	7.6	7.3	7.0	6.7	6.4	6.2	5.9	5.6				
3.0	10.1	9.8	9.6	9.3	9.0	8.7	8.4	8.2	7.9	7.6	7.3	7.0	6.8	6.5	6.2	5.9	5.6			
3.1	10.3	10.0	9.8	9.5	9.2	8.9	8.7	8.4	8.1	7.9	7.6	7.3	7.0	6.8	6.5	6.2	6.0	5.7		
3.2	10.5	10.2	10.0	9.7	9.4	9.2	8.9	8.6	8.4	8.1	7.9	7.6	7.3	7.1	6.8	6.6	6.3	6.0	5.7	5.5
3.3	10.6	10.4	10.1	9.9	9.6	9.4	9.1	8.9	8.6	8.4	8.1	7.8	7.6	7.3	7.1	6.8	6.6	6.3	6.1	5.8
3.4	10.8	10.6	10.3	10.1	9.8	9.6	9.3	9.1	8.8	8.6	8.3	8.1	7.8	7.6	7.4	7.1	6.8	6.6	6.4	6.1
3.5	11.0	10.7	10.5	10.2	10.0	9.8	9.5	9.3	9.0	8.8	8.6	8.3	8.1	7.8	7.6	7.4	7.1	6.9	6.6	6.4
3.6	11.1	10.9	10.6	10.4	10.2	9.9	9.7	9.5	9.2	9.0	8.8	8.5	8.3	8.1	7.8	7.6	7.4	7.1	6.9	6.7
3.7	11.2	11.0	10.8	10.5	10.3	10.1	9.9	9.6	9.4	9.2	9.0	8.7	8.5	8.3	8.1	7.8	7.6	7.4	7.1	6.9
3.8	11.4	11.1	10.9	10.7	10.5	10.2	10.0	9.8	9.6	9.4	9.1	8.9	8.7	8.5	8.3	8.0	7.8	7.6	7.4	7.2
3.9	11.5	11.3	11.0	10.8	10.6	10.4	10.2	10.0	9.7	9.5	9.3	9.1	8.9	8.7	8.5	8.2	8.0	7.8	7.6	7.4
4.0	11.6	11.4	11.2	11.0	10.7	10.5	10.3	10.1	9.9	9.7	9.5	9.3	9.1	8.9	8.6	8.4	8.2	8.0	7.8	7.6
4.1	11.7	11.5	11.3	11.1	10.9	10.7	10.5	10.3	10.1	9.8	9.6	9.4	9.2	9.0	8.8	8.6	8.4	8.2	8.0	7.8
4.2	11.8	11.6	11.4	11.2	11.0	10.8	10.6	10.4	10.2	10.0	9.8	9.6	9.4	9.2	9.0	8.8	8.6	8.4	8.2	8.0
4.3	11.9	11.7	11.5	11.3	11.1	10.9	10.7	10.5	10.3	10.1	9.9	9.7	9.5	9.4	9.2	9.0	8.8	8.6	8.4	8.2
4.4	12.0	11.8	11.6	11.4	11.2	11.0	10.8	10.7	10.5	10.3	10.1	9.9	9.7	9.5	9.3	9.1	8.9	8.7	8.5	8.4
4.5	12.1	11.9	11.7	11.5	11.3	11.1	11.0	10.8	10.6	10.4	10.2	10.0	9.8	9.6	9.5	9.3	9.1	8.9	8.7	8.5
4.6	12.2	12.0	11.8	11.6	11.4	11.2	11.1	10.9	10.7	10.5	10.3	10.2	10.0	9.8	9.6	9.4	9.2	9.1	8.9	8.7
4.7	12.2	12.1	11.9	11.7	11.5	11.3	11.2	11.0	10.8	10.6	10.5	10.3	10.1	9.9	9.7	9.6	9.4	9.2	9.0	8.8
4.8	12.3	12.1	12.0	11.8	11.6	11.4	11.3	11.1	10.9	10.7	10.6	10.4	10.2	10.0	9.9	9.7	9.5	9.3	9.2	9.0
4.9	12.4	12.2	12.1	11.9	11.7	11.5	11.4	11.2	11.0	10.9	10.7	10.5	10.3	10.2	10.0	9.8	9.7	9.5	9.3	9.1
5.0	12.5	12.3	12.1	12.0	11.8	11.6	11.5	11.3	11.1	11.0	10.8	10.6	10.4	10.3	10.1	9.9	9.8	9.6	9.4	9.3
5.1	12.5	12.4	12.2	12.0	11.9	11.7	11.5	11.4	11.2	11.1	10.9	10.7	10.6	10.4	10.2	10.1	9.9	9.7	9.6	9.4
5.2	12.6	12.4	12.3	12.1	12.0	11.8	11.6	11.5	11.3	11.1	11.0	10.8	10.7	10.5	10.3	10.2	10.0	9.9	9.7	9.5
5.3	12.7	12.5	12.3	12.2	12.0	11.9	11.7	11.6	11.4	11.2	11.1	10.9	10.8	10.6	10.4	10.3	10.1	10.0	9.8	9.7
5.4	12.7	12.6	12.4	12.3	12.1	12.0	11.8	11.6	11.5	11.3	11.2	11.0	10.9	10.7	10.5	10.4	10.2	10.1	9.9	9.8
5.5	12.8	12.6	12.5	12.3	12.2	12.0	11.9	11.7	11.6	11.4	11.3	11.1	11.0	10.8	10.6	10.5	10.3	10.2	10.0	9.9
5.6	12.8	12.7	12.5	12.4	12.2	12.1	11.9	11.8	11.6	11.5	11.3	11.2	11.0	10.9	10.7	10.6	10.4	10.3	10.1	10.0
5.7	12.9	12.8	12.6	12.5	12.3	12.2	12.0	11.9	11.7	11.6	11.4	11.3	11.1	11.0	10.8	10.7	10.5	10.4	10.2	10.1
5.8	13.0	12.8	12.7	12.5	12.4	12.2	12.1	11.9	11.8	11.7	11.5	11.4	11.2	11.1	10.9	10.8	10.6	10.5	10.3	10.2
5.9	13.0	12.9	12.7	12.6	12.4	12.3	12.2	12.0	11.9	11.7	11.6	11.4	11.3	11.2	11.0	10.9	10.7	10.6	10.4	10.3
6.0	13.1	12.9	12.8	12.6	12.5	12.4	12.2	12.1	11.9	11.8	11.7	11.5	11.4	11.2	11.1	11.0	10.8	10.7	10.5	10.4

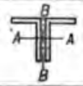
r	41	42	43	44	45	46	47	48	r
0.3									0.3
0.4									0.4
0.5									0.5
0.6									0.6
0.7									0.7
0.8									0.8
0.9									0.9
1.0									1.0
1.1									1.1
1.2									1.2
1.3									1.3
1.4									1.4
1.5									1.5
1.6									1.6
1.7									1.7
1.8									1.8
1.9									1.9
2.0									2.0
2.1									2.1
2.2									2.2
2.3									2.3
2.4									2.4
2.5									2.5
2.6									2.6
2.7									2.7
2.8									2.8
2.9									2.9
3.0									3.0
3.1									3.1
3.2									3.2
3.3	5.6								3.3
3.4	5.9	5.6							3.4
3.5	6.2	5.9	5.7						3.5
3.6	6.4	6.2	6.0	5.7	5.5				3.6
3.7	6.7	6.5	6.2	6.0	5.8	5.6			3.7
3.8	6.9	6.7	6.5	6.3	6.0	5.8	5.6		3.8
3.9	7.2	6.9	6.7	6.5	6.3	6.1	5.9	5.7	3.9
4.0	7.4	7.2	7.0	6.8	6.6	6.3	6.1	5.9	4.0
4.1	7.6	7.4	7.2	7.0	6.8	6.6	6.4	6.2	4.1
4.2	7.8	7.6	7.4	7.2	7.0	6.8	6.6	6.4	4.2
4.3	8.0	7.8	7.6	7.4	7.2	7.0	6.8	6.6	4.3
4.4	8.2	8.0	7.8	7.6	7.4	7.2	7.0	6.8	4.4
4.5	8.3	8.2	8.0	7.8	7.6	7.4	7.2	7.0	4.5
4.6	8.5	8.3	8.1	8.0	7.8	7.6	7.4	7.2	4.6
4.7	8.7	8.5	8.3	8.1	7.9	7.8	7.6	7.4	4.7
4.8	8.8	8.6	8.5	8.3	8.1	7.9	7.8	7.6	4.8
4.9	9.0	8.8	8.6	8.4	8.3	8.1	7.9	7.8	4.9
5.0	9.1	8.9	8.8	8.6	8.4	8.3	8.1	7.9	5.0
5.1	9.2	9.1	8.9	8.7	8.6	8.4	8.3	8.1	5.1
5.2	9.4	9.2	9.1	8.9	8.7	8.6	8.4	8.2	5.2
5.3	9.5	9.3	9.2	9.0	8.9	8.7	8.6	8.4	5.3
5.4	9.6	9.5	9.3	9.2	9.0	8.8	8.7	8.5	5.4
5.5	9.7	9.6	9.4	9.3	9.1	9.0	8.8	8.7	5.5
5.6	9.8	9.7	9.5	9.4	9.2	9.1	9.0	8.8	5.6
5.7	10.0	9.8	9.7	9.5	9.4	9.2	9.1	8.9	5.7
5.8	10.1	9.9	9.8	9.6	9.5	9.3	9.2	9.0	5.8
5.9	10.2	10.0	9.9	9.7	9.6	9.4	9.3	9.2	5.9
6.0	10.3	10.1	10.0	9.8	9.7	9.6	9.4	9.3	6.0

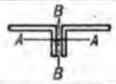
RADII OF GYRATION IN INCHES FOR TWO ANGLES ARRANGED AS SHOWN

EQUAL LEGS

SIZE	THICKNESS	AREA 2 Ls	AXIS AA	DISTANCE B TO B. OF ANGLES—AXIS BB: 0	5/16	3/8	7/16	1/2	9/16	5/8
8×8	1 1/8	33.46	2.42	3.42	3.53	3.55	3.57	3.60	3.62	3.64
	1 1/16	31.74	2.43	3.41	3.52	3.54	3.56	3.59	3.61	3.63
	1	30.00	2.44	3.40	3.51	3.53	3.55	3.58	3.60	3.62
	15/16	28.24	2.44	3.38	3.49	3.52	3.54	3.56	3.58	3.61
	7/8	26.46	2.45	3.38	3.48	3.51	3.53	3.55	3.57	3.60
	13/16	24.68	2.46	3.37	3.48	3.50	3.52	3.54	3.57	3.59
	3/4	22.88	2.47	3.36	3.47	3.49	3.51	3.54	3.56	3.58
	11/16	21.06	2.48	3.35	3.46	3.48	3.50	3.52	3.54	3.57
	5/8	19.22	2.49	3.34	3.45	3.47	3.49	3.51	3.54	3.56
	9/16	17.36	2.50	3.33	3.44	3.46	3.48	3.50	3.53	3.55
	1/2	15.50	2.51	3.33	3.44	3.45	3.48	3.50	3.52	3.54
6×6	1	22.00	1.80	2.59	2.70	2.72	2.75	2.77	2.80	2.82
	15/16	20.74	1.80	2.58	2.69	2.71	2.74	2.76	2.78	2.81
	7/8	19.46	1.81	2.57	2.68	2.70	2.73	2.75	2.77	2.80
	13/16	18.18	1.82	2.56	2.67	2.69	2.72	2.74	2.76	2.79
	3/4	16.88	1.83	2.55	2.66	2.68	2.71	2.73	2.76	2.78
	11/16	15.56	1.83	2.53	2.64	2.67	2.69	2.71	2.74	2.76
	5/8	14.22	1.84	2.53	2.64	2.66	2.68	2.71	2.73	2.75
	9/16	12.86	1.85	2.52	2.63	2.65	2.67	2.70	2.72	2.74
	1/2	11.50	1.86	2.51	2.62	2.64	2.66	2.68	2.70	2.73
	7/16	10.12	1.87	2.50	2.61	2.63	2.65	2.67	2.69	2.72
	3/8	8.72	1.88	2.49	2.60	2.62	2.64	2.66	2.69	2.71
4×4	3/4	10.88	1.19	1.74	1.86	1.88	1.91	1.93	1.95	1.98
	11/16	10.06	1.19	1.73	1.84	1.87	1.89	1.92	1.94	1.96
	5/8	9.22	1.20	1.72	1.83	1.86	1.88	1.91	1.93	1.95
	9/16	8.36	1.21	1.71	1.83	1.85	1.87	1.90	1.92	1.94
	1/2	7.50	1.22	1.70	1.81	1.83	1.86	1.88	1.90	1.93
	7/16	6.62	1.23	1.69	1.80	1.82	1.85	1.87	1.89	1.92
	3/8	5.72	1.23	1.68	1.79	1.81	1.84	1.86	1.88	1.90
	5/16	4.80	1.24	1.67	1.78	1.80	1.83	1.85	1.87	1.89
3½×3½	5/8	7.96	1.04	1.52	1.63	1.66	1.68	1.70	1.73	1.75
	9/16	7.24	1.05	1.51	1.62	1.65	1.67	1.69	1.72	1.74
	1/2	6.50	1.06	1.50	1.61	1.64	1.66	1.68	1.71	1.73
	7/16	5.74	1.07	1.49	1.60	1.63	1.65	1.67	1.70	1.72
	3/8	4.96	1.07	1.48	1.59	1.61	1.63	1.66	1.68	1.70
	5/16	4.18	1.08	1.47	1.58	1.60	1.62	1.65	1.67	1.69
3×3	1/2	5.50	0.90	1.29	1.41	1.43	1.46	1.48	1.51	1.53
	7/16	4.86	0.91	1.28	1.40	1.42	1.45	1.47	1.50	1.52
	3/8	4.22	0.91	1.27	1.39	1.41	1.44	1.46	1.48	1.51
	5/16	3.56	0.92	1.27	1.38	1.40	1.43	1.45	1.47	1.50
	1/4	2.88	0.93	1.25	1.36	1.39	1.41	1.43	1.46	1.48
2½×2½	7/16	4.00	0.75	1.08	1.20	1.22	1.25	1.27	1.30	1.32
	3/8	3.46	0.75	1.07	1.18	1.21	1.23	1.26	1.28	1.31
	5/16	2.94	0.76	1.06	1.18	1.20	1.22	1.25	1.27	1.30
	1/4	2.38	0.77	1.05	1.17	1.19	1.21	1.24	1.26	1.29
	3/16	1.80	0.78	1.04	1.15	1.18	1.20	1.22	1.25	1.27
2×2	3/8	2.72	0.59	0.87	0.99	1.02	1.04	1.07	1.09	1.12
	5/16	2.30	0.60	0.86	0.98	1.00	1.02	1.05	1.07	1.10
	1/4	1.88	0.61	0.85	0.96	0.99	1.01	1.04	1.06	1.09
	3/16	1.42	0.62	0.84	0.95	0.98	1.00	1.03	1.05	1.08

UNEQUAL LEGS





SIZE	THICKNESS	AREA 2 Ls	AXIS AA	DISTANCE B. TO B. OF ANGLES — AXIS BB: 0	5/16	3/8	7/16	1/2	9/16	5/8	AXIS AA	DISTANCE B. TO B. OF ANGLES — AXIS BB: 0	5/16	3/8	7/16	1/2	9/16	5/8
6×4	7/8	15.96	1.86	1.58	1.69	1.71	1.74	1.76	1.79	1.81	1.11	2.82	2.94	2.97	2.99	3.01	3.04	3.06
	13/16	14.94	1.87	1.56	1.68	1.70	1.73	1.75	1.77	1.80	1.11	2.81	2.93	2.96	2.98	3.00	3.03	3.05
	3/4	13.88	1.88	1.56	1.67	1.69	1.72	1.74	1.76	1.79	1.12	2.80	2.92	2.95	2.97	2.99	3.02	3.04
	11/16	12.80	1.89	1.55	1.66	1.68	1.71	1.73	1.75	1.77	1.13	2.79	2.91	2.94	2.96	2.98	3.01	3.03
	5/8	11.72	1.90	1.53	1.64	1.66	1.69	1.71	1.73	1.76	1.13	2.78	2.89	2.92	2.94	2.97	2.99	3.01
	9/16	10.62	1.90	1.52	1.63	1.65	1.68	1.70	1.72	1.75	1.14	2.77	2.88	2.91	2.93	2.95	2.98	3.00
	1/2	9.50	1.91	1.52	1.62	1.65	1.67	1.69	1.71	1.74	1.15	2.76	2.88	2.90	2.92	2.95	2.97	2.99
	7/16	8.36	1.92	1.50	1.61	1.63	1.65	1.68	1.70	1.72	1.16	2.75	2.86	2.88	2.91	2.93	2.95	2.98
	3/8	7.22	1.93	1.50	1.60	1.62	1.64	1.67	1.69	1.71	1.17	2.74	2.85	2.87	2.90	2.92	2.94	2.97
6×3½	7/8	15.10	1.87	1.34	1.46	1.49	1.51	1.53	1.56	1.58	0.93	2.90	3.02	3.05	3.07	3.10	3.12	3.15
	13/16	14.12	1.88	1.34	1.45	1.48	1.50	1.52	1.55	1.57	0.94	2.89	3.01	3.04	3.06	3.09	3.11	3.14
	3/4	13.12	1.89	1.32	1.44	1.46	1.48	1.51	1.53	1.56	0.94	2.88	3.00	3.03	3.05	3.08	3.10	3.13
	11/16	12.12	1.89	1.31	1.42	1.45	1.47	1.49	1.52	1.54	0.95	2.86	2.98	3.01	3.03	3.06	3.08	3.11
	5/8	11.10	1.90	1.30	1.41	1.43	1.46	1.48	1.51	1.53	0.96	2.85	2.97	3.00	3.02	3.05	3.07	3.10
	9/16	10.06	1.91	1.29	1.40	1.42	1.44	1.47	1.49	1.52	0.96	2.85	2.96	2.99	3.01	3.04	3.06	3.09
	1/2	9.00	1.92	1.28	1.38	1.41	1.43	1.45	1.48	1.50	0.97	2.83	2.95	2.97	3.00	3.02	3.04	3.07
	7/16	7.94	1.93	1.27	1.37	1.40	1.42	1.44	1.47	1.49	0.98	2.82	2.94	2.96	2.99	3.01	3.03	3.06
	3/8	6.84	1.94	1.26	1.36	1.39	1.41	1.43	1.45	1.47	0.99	2.81	2.93	2.95	2.98	3.00	3.02	3.05
5×3½	3/4	11.62	1.55	1.40	1.51	1.54	1.56	1.59	1.61	1.63	0.98	2.34	2.46	2.48	2.51	2.53	2.55	2.58
	11/16	10.74	1.56	1.38	1.49	1.52	1.54	1.57	1.59	1.61	0.98	2.32	2.44	2.46	2.49	2.51	2.53	2.56
	5/8	9.84	1.56	1.37	1.49	1.51	1.53	1.56	1.58	1.60	0.99	2.31	2.43	2.45	2.48	2.50	2.52	2.55
	9/16	8.94	1.57	1.37	1.48	1.50	1.52	1.55	1.57	1.59	1.00	2.30	2.42	2.44	2.46	2.49	2.51	2.54
	1/2	8.00	1.58	1.36	1.47	1.49	1.51	1.54	1.56	1.58	1.01	2.29	2.41	2.43	2.45	2.48	2.50	2.53
	7/16	7.06	1.59	1.34	1.45	1.47	1.50	1.52	1.54	1.57	1.01	2.28	2.39	2.42	2.44	2.46	2.49	2.51
	3/8	6.10	1.60	1.34	1.44	1.46	1.49	1.51	1.53	1.55	1.02	2.27	2.38	2.41	2.43	2.45	2.48	2.50
	5/16	5.12	1.61	1.33	1.43	1.45	1.48	1.50	1.52	1.55	1.03	2.26	2.37	2.40	2.42	2.44	2.47	2.49
5×3	11/16	10.06	1.56	1.15	1.27	1.29	1.32	1.34	1.37	1.39	0.81	2.40	2.52	2.54	2.57	2.59	2.62	2.64
	5/8	9.22	1.57	1.14	1.25	1.28	1.30	1.33	1.35	1.38	0.81	2.39	2.51	2.53	2.56	2.58	2.61	2.63
	9/16	8.36	1.58	1.12	1.24	1.26	1.29	1.31	1.33	1.36	0.82	2.37	2.49	2.52	2.54	2.57	2.59	2.61
	1/2	7.50	1.59	1.12	1.23	1.25	1.28	1.30	1.32	1.35	0.83	2.36	2.48	2.50	2.53	2.55	2.58	2.60
	7/16	6.62	1.60	1.11	1.22	1.24	1.27	1.29	1.31	1.34	0.84	2.36	2.47	2.49	2.52	2.54	2.57	2.59
	3/8	5.72	1.61	1.10	1.21	1.23	1.25	1.27	1.29	1.32	0.84	2.34	2.46	2.48	2.51	2.53	2.55	2.58
	5/16	4.80	1.61	1.09	1.19	1.22	1.24	1.26	1.28	1.31	0.85	2.33	2.44	2.47	2.49	2.52	2.54	2.56
4×3	5/8	7.96	1.23	1.22	1.33	1.36	1.38	1.41	1.43	1.46	0.85	1.84	1.96	1.98	2.01	2.03	2.06	2.08
	9/16	7.24	1.24	1.21	1.32	1.35	1.37	1.40	1.42	1.45	0.86	1.83	1.95	1.97	2.00	2.02	2.05	2.07
	1/2	6.50	1.25	1.20	1.31	1.33	1.36	1.38	1.40	1.43	0.86	1.82	1.94	1.96	1.99	2.01	2.04	2.06
	7/16	5.74	1.25	1.18	1.29	1.32	1.34	1.36	1.39	1.41	0.87	1.81	1.92	1.95	1.97	1.99	2.02	2.04
	3/8	4.96	1.26	1.18	1.28	1.31	1.33	1.35	1.38	1.40	0.88	1.80	1.91	1.94	1.96	1.98	2.01	2.03
	5/16	4.18	1.27	1.17	1.28	1.30	1.32	1.35	1.37	1.39	0.89	1.79	1.90	1.93	1.95	1.97	2.00	2.02
3½×3	9/16	6.68	1.07	1.25	1.37	1.39	1.42	1.44	1.47	1.49	0.87	1.57	1.69	1.71	1.74	1.76	1.79	1.81
	1/2	6.00	1.07	1.24	1.36	1.38	1.41	1.43	1.46	1.48	0.88	1.56	1.67	1.70	1.72	1.75	1.77	1.80
	7/16	5.30	1.08	1.23	1.34	1.37	1.39	1.41	1.44	1.46	0.89	1.54	1.66	1.68	1.71	1.73	1.75	1.78
	3/8	4.60	1.09	1.22	1.33	1.36	1.38	1.40	1.43	1.45	0.90	1.53	1.65	1.67	1.70	1.72	1.74	1.77
	5/16	3.86	1.10	1.21	1.32	1.35	1.37	1.39	1.41	1.43	0.90	1.52	1.64	1.66	1.69	1.71	1.73	1.76
3½×2½	1/2	5.50	1.09	0.99	1.11	1.13	1.16	1.18	1.20	1.23	0.70	1.62	1.74	1.76	1.79	1.81	1.84	1.86
	7/16	4.86	1.09	0.98	1.10	1.12	1.15	1.17	1.19	1.22	0.71	1.61	1.73	1.75	1.77	1.80	1.82	1.85
	3/8	4.22	1.10	0.97	1.09	1.11	1.14	1.16	1.18	1.21	0.72	1.60	1.72	1.74	1.76	1.79	1.81	1.84
	5/16	3.56	1.11	0.97	1.08	1.10	1.13	1.15	1.17	1.20	0.73	1.59	1.71	1.73	1.75	1.78	1.80	1.83
	1/4	2.88	1.12	0.96	1.06	1.09	1.11	1.13	1.16	1.18	0.74	1.58	1.69	1.71	1.74	1.76	1.79	1.81
3×2½	7/16	4.42	0.92	1.03	1.15	1.17	1.20	1.22	1.25	1.27	0.73	1.34	1.46	1.49	1.51	1.54	1.56	1.59
	3/8	3.84	0.93	1.02	1.14	1.16	1.19	1.21	1.24	1.26	0.74	1.34	1.45	1.48	1.50	1.53	1.55	1.58
	5/16	3.24	0.94	1.01	1.12	1.14	1.17	1.19	1.21	1.24	0.74	1.32	1.43	1.46	1.48	1.51	1.53	1.56
	1/4	2.62	0.95	1.00	1.11	1.13	1.16	1.18	1.20	1.23	0.75	1.31	1.42	1.45	1.47	1.50	1.52	1.55
2½×2	3/8	3.10	0.77	0.82	0.94	0.96	0.98	1.01	1.04	1.06	0.58	1.13	1.25	1.28	1.30	1.33	1.35	1.38
	5/16	2.62	0.78	0.81	0.93	0.95	0.97	1.00	1.02	1.05	0.58	1.12	1.24	1.26	1.29	1.32	1.34	1.37
	1/4	2.12	0.78	0.80	0.91	0.94	0.96	0.99	1.01	1.04	0.59	1.11	1.23	1.25	1.28	1.30	1.32	1.35
	3/16	1.62	0.79	0.79	0.90	0.92	0.94	0.97	0.99	1.02	0.60	1.10	1.21	1.24	1.26	1.29	1.31	1.33

SINGLE ANGLE STRUTS—SAFE LOADS

l = LENGTH OF MEMBER IN INCHES
r = LEAST RADIUS OF GYRATION IN INCHES
* UNIT STRESS EXCEEDS 14,000 POUNDS

SAFE LOADS IN THOUSANDS OF POUNDS AT $16{,}000 - 70\,l/r$

FOR VALUES TO THE LEFT OF THE HEAVY LINES THE RATIO l/r DOES NOT EXCEED 100
" " BETWEEN " " " " " " " " " 120
" " " THE RIGHT OF " " " " " " " " " 150

SIZE	THICKNESS	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
		LENGTH OF MEMBER IN FEET																
8×8	1 1/8	240*	232	222	213	204	195	186	177	168	159	150	141	132	123	113	104	95
	1 1/16	228*	220	211	202	194	185	177	168	160	151	143	134	126	117	109	100	92
	1	216*	208	200	191	184	175	167	159	151	143	135	127	119	111	103	95	87
	15/16	203*	196	188	180	173	165	157	150	142	135	127	119	112	104	97	89	82
	7/8	190*	183	176	169	162	155	148	140	133	126	119	112	105	98	91	83	76
	13/16	178*	171	165	158	151	145	138	131	125	118	112	105	98	92	85	78	72
	3/4	165*	159	153	146	140	134	128	122	116	110	103	97	91	85	79	73	67
	11/16	152*	146	141	135	129	124	118	112	107	101	96	90	85	79	73	68	62
	5/8	138*	133	128	123	118	113	108	103	98	92	87	82	77	72	67	62	57
	9/16	125*	120	116	111	107	102	97	93	88	84	79	74	70	65	61	56	51
	1/2	112*	107	103	99	95	91	87	83	79	75	70	66	62	58	54	50	46
6×6	1	152	144	136	128	120	112	104	96	88	80	72	64	....	....	....	....	....
	15/16	144	136	128	121	113	106	98	91	83	76	68	61	....	....	....	....	....
	7/8	135	128	121	114	107	100	93	86	79	72	65	58	....	....	....	....	....
	13/16	126	119	113	106	100	93	87	80	74	67	61	54	....	....	....	....	....
	3/4	117	111	105	99	93	87	80	74	68	62	56	50	....	....	....	....	....
	11/16	108	102	97	91	85	80	74	69	63	57	52	46	....	....	....	....	....
	5/8	98	93	88	83	78	73	68	63	58	52	47	42	....	....	....	....	....
	9/16	89	85	80	75	71	66	62	57	53	48	43	39	....	....	....	....	....
	1/2	80	76	71	67	63	59	55	51	47	43	39	35	....	....	....	....	....
	7/16	70	67	63	59	56	52	49	45	42	38	34	31	....	....	....	....	....
	3/8	60	57	54	51	48	45	42	39	36	33	30	27	....	....	....	....	....
4×4	3/4	69	63	57	51	45	40	34	....	....	....	....	....	....	....	....	....	....
	11/16	64	59	53	48	42	37	31	....	....	....	....	....	....	....	....	....	....
	5/8	59	54	49	44	39	34	28	....	....	....	....	....	....	....	....	....	....
	9/16	53	49	44	40	35	31	26	....	....	....	....	....	....	....	....	....	....
	1/2	48	44	40	36	32	28	24	....	....	....	....	....	....	....	....	....	....
	7/16	42	39	35	32	28	24	21	....	....	....	....	....	....	....	....	....	....
	3/8	37	34	31	27	24	21	18	....	....	....	....	....	....	....	....	....	....
	5/16	31	28	26	23	21	18	15	....	....	....	....	....	....	....	....	....	....
3½×3½	5/8	49	44	39	34	29	24	....	....	....	....	....	....	....	....	....	....	....
	9/16	44	40	36	31	27	22	....	....	....	....	....	....	....	....	....	....	....
	1/2	40	36	32	28	24	20	....	....	....	....	....	....	....	....	....	....	....
	7/16	35	32	28	25	21	18	....	....	....	....	....	....	....	....	....	....	....
	3/8	31	28	25	22	19	16	....	....	....	....	....	....	....	....	....	....	....
	5/16	26	23	21	18	16	13	....	....	....	....	....	....	....	....	....	....	....
3×3	1/2	32	28	24	20	16	....	....	....	....	....	....	....	....	....	....	....	....
	7/16	28	25	21	18	14	....	....	....	....	....	....	....	....	....	....	....	....
	3/8	25	22	18	15	12	....	....	....	....	....	....	....	....	....	....	....	....
	5/16	21	18	16	13	11	....	....	....	....	....	....	....	....	....	....	....	....
	1/4	17	15	13	11	9	....	....	....	....	....	....	....	....	....	....	....	....
2½×2½	7/16	21	18	14	11	....	....	....	....	....	....	....	....	....	....	....	....	....
	3/8	19	16	13	10	....	....	....	....	....	....	....	....	....	....	....	....	....
	5/16	16	13	11	8	....	....	....	....	....	....	....	....	....	....	....	....	....
	1/4	13	11	9	7	....	....	....	....	....	....	....	....	....	....	....	....	....
	3/16	10	8	7	5	....	....	....	....	....	....	....	....	....	....	....	....	....
2×2	3/8	13	10	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
	5/16	11	8	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
	1/4	9	7	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
	3/16	7	5	4	....	....	....	....	....	....	....	....	....	....	....	....	....	....
SIZE	THICKNESS	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
		LENGTH OF MEMBER IN FEET																

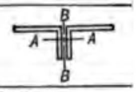
SIZE	THICKNESS	3	4	5	6	7	8	9	10	11
		LENGTH OF MEMBER IN FEET								
6×4	7/8	104	96	89	81	73	65	58	50	....
	13/16	98	90	83	76	68	61	54	47	....
	3/4	91	84	77	70	64	57	50	43	....
	11/16	84	77	71	65	59	52	46	40	....
	5/8	77	71	65	59	54	48	42	36	....
	9/16	70	64	59	54	49	44	39	34	....
	1/2	62	58	53	49	44	39	35	30	....
	7/16	55	51	47	43	39	35	31	27	....
	3/8	47	44	41	37	34	30	27	23	20
6×3½	7/8	95	87	79	70	62	53	45	....	....
	13/16	89	81	73	66	58	50	42	....	....
	3/4	83	76	68	61	54	46	39	....	....
	11/16	77	70	63	56	49	43	36	....	....
	5/8	70	64	58	51	45	39	33	....	....
	9/16	64	58	52	47	41	35	30	....	....
	1/2	57	52	47	42	37	32	27	....	....
	7/16	50	46	42	37	33	28	24	....	....
	3/8	44	40	36	32	29	25	21	....	....
5×3½	3/4	73	67	60	54	47	41	34	....	....
	11/16	68	62	56	50	44	38	32	....	....
	5/8	62	57	51	46	40	35	29	....	....
	9/16	56	52	46	41	36	31	26	....	....
	1/2	51	46	42	37	33	28	24	....	....
	7/16	45	41	37	33	29	25	21	....	....
	3/8	39	35	32	29	25	22	18	....	....
	5/16	32	30	27	24	21	18	15	....	....
5×3	11/16	61	54	47	41	34	28	....	....	....
	5/8	56	50	43	37	31	25	....	....	....
	9/16	51	45	40	34	29	24	....	....	....
	1/2	45	41	36	31	26	21	....	....	....
	7/16	40	36	32	27	23	19	....	....	....
	3/8	35	31	27	24	20	16	....	....	....
	5/16	29	26	23	20	17	14	....	....	....
4×3	5/8	48	43	38	32	27	22	....	....	....
	9/16	44	39	34	29	25	20	....	....	....
	1/2	39	35	31	26	22	18	....	....	....
	7/16	35	31	27	23	20	16	....	....	....
	3/8	30	27	23	20	17	14	....	....	....
	5/16	25	23	20	17	15	12	....	....	....
3½×3	9/16	40	35	31	26	22	....	....	....	....
	1/2	36	32	28	24	20	....	....	....	....
	7/16	32	28	24	21	17	....	....	....	....
	3/8	27	24	21	18	15	....	....	....	....
	5/16	23	21	18	15	13	....	....	....	....
3½×2½	1/2	31	27	22	18	....	....	....	....	....
	7/16	28	24	20	16	....	....	....	....	....
	3/8	24	21	17	14	....	....	....	....	....
	5/16	20	17	15	12	....	....	....	....	....
	1/4	16	14	12	10	....	....	....	....	....
3×2½	7/16	25	21	18	14	....	....	....	....	....
	3/8	21	18	15	12	....	....	....	....	....
	5/16	18	16	13	11	....	....	....	....	....
	1/4	15	13	11	8	....	....	....	....	....
2½×2	3/8	15	12	9	....	....	....	....	....	....
	5/16	13	10	8	....	....	....	....	....	....
	1/4	11	8	6	....	....	....	....	....	....
	3/16	8	7	5	....	....	....	....	....	....
SIZE	THICKNESS	3	4	5	6	7	8	9	10	11
		LENGTH OF MEMBER IN FEET								

TWO ANGLE STRUTS—SAFE LOADS

SAFE LOADS IN THOUSANDS OF POUNDS AT 16,000 – 70 l/r. FOR EXPLANATIONS SEE PRECEDING PAGE

LEAST RADIUS ABOUT AXIS BB FOR 5×3 Ls OR OVER, AND 3½×2½×¼ AND 3½×2½×5/16
LEAST RADIUS ABOUT AXIS AA FOR 4×3 Ls OR UNDER EXCEPT 3½×2½×¼ AND 3½×2½×5/16
ANY DISTANCE APART FOR 3½×3, 3×2½ AND 2½×2
3/8" B. TO B. FOR ALL OTHER ANGLES

LEAST RADIUS ABOUT AXIS AA
ANY DISTANCE APART



Size	Thickness	Length of member in feet (least radius about axis BB / AA, as above)																			Length of member in feet (least radius about axis AA, any distance apart)										
		3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	3	4	5	6	7	8	9	10	11	12	13
6×4	7/8	232*	224*	216	208	200	193	185	177	169	161	153	146	138	130	122	114	106	99	91	219	207	195	183	171	159	147	134	122	110	98
	13/16	217*	210*	202	195	187	180	173	165	158	150	143	136	128	121	114	106	99	91	84	206	194	183	171	160	149	137	126	115	103	92
	3/4	201*	195*	188	181	174	167	160	153	146	139	132	125	118	112	105	98	91	84	77	191	180	170	160	149	139	128	118	108	97	87
	11/16	186*	179*	173	166	160	154	147	141	134	128	122	115	109	102	96	90	83	77	70	176	167	157	148	138	129	119	110	100	91	81
	5/8	170*	164	158	152	146	140	134	128	122	116	110	104	98	93	87	81	75	69	...	161	153	144	135	127	118	109	100	92	83	74
	9/16	154*	148	143	138	132	127	121	116	110	105	100	94	89	84	78	73	67	62	...	146	139	131	123	115	107	100	92	84	76	68
	1/2	138*	133	128	123	118	113	109	104	99	94	89	84	79	75	70	65	60	55	...	131	124	117	110	103	96	90	83	76	69	62
	7/16	121*	117	112	108	104	99	95	91	86	82	78	74	69	65	61	56	52	48	...	116	110	104	97	91	85	79	73	67	61	55
	3/8	104*	101	97	93	89	86	82	78	74	71	67	63	59	56	52	48	45	41	...	100	95	90	84	79	74	69	64	58	53	48
6×3½	7/8	216*	208	199	191	182	173	165	156	148	139	131	122	114	105	97	88	...	...	...	200	187	173	160	146	132	119	105	91	...	...
	13/16	202*	194	186	178	170	162	154	146	138	130	122	114	106	98	90	82	...	...	...	188	175	163	150	138	125	112	100	87	...	...
	3/4	187*	180	172	165	157	150	142	134	127	119	112	104	97	89	82	74	...	...	...	175	163	151	140	128	116	104	93	81	...	...
	11/16	173*	166	159	152	145	138	131	124	117	110	103	96	89	82	75	68	...	...	...	162	151	140	130	119	108	97	87	76	...	...
	5/8	158*	152	145	138	132	125	119	112	106	99	93	86	80	73	67	...	...	...	...	148	139	129	119	110	100	90	80	71	61	...
	9/16	143*	137	131	125	119	113	107	101	95	90	84	78	72	66	60	...	...	...	...	134	126	117	108	99	90	82	73	64	55	...
	1/2	128*	123	117	112	106	101	96	90	85	80	74	69	64	58	53	...	...	...	...	121	113	105	97	89	82	74	66	58	50	...
	7/16	113*	108	103	98	94	89	84	79	75	70	65	60	56	51	46	...	...	...	...	107	100	93	86	79	73	66	59	52	45	...
	3/8	97*	93	89	85	81	76	72	68	64	60	56	52	47	43	39	...	...	...	...	92	86	80	75	69	63	57	51	46	40	...
5×3½	3/4	167*	161	154	148	142	135	129	123	116	110	103	97	91	85	78	72	66	...	...	156	146	136	126	116	106	96	86	76	66	...
	11/16	154*	148	142	136	130	124	118	113	107	101	95	89	83	77	71	65	59	...	...	144	135	126	117	107	98	89	80	71	61	...
	5/8	141*	136	130	125	119	114	108	103	97	92	86	81	75	70	64	59	...	...	...	132	124	116	107	99	91	82	74	66	57	...
	9/16	128*	123	118	113	108	103	98	93	88	83	78	73	68	63	58	53	...	...	...	121	113	105	98	90	83	75	68	60	53	...
	1/2	114*	110	105	101	96	92	87	83	78	74	69	65	60	56	51	47	...	...	...	108	101	95	88	81	75	68	61	55	48	...
	7/16	101*	97	93	89	85	81	77	73	69	65	61	56	52	48	44	40	...	...	...	95	89	84	78	72	66	60	54	48	42	...
	3/8	87*	84	80	77	73	70	66	62	59	55	52	48	45	41	38	34	...	...	...	83	78	72	67	62	57	52	47	42	37	...
	5/16	73*	70	67	64	61	58	55	52	49	46	43	40	37	34	31	29	...	...	...	69	65	61	57	53	48	44	40	36	32	...
5×3	11/16	141*	135	128	122	115	109	102	95	89	82	76	69	63	56	...	...	...	...	...	130	119	109	98	88	77	67	57	...	...	...
	5/8	129*	123	117	111	105	99	93	87	81	75	69	63	57	51	...	...	...	...	...	119	109	100	90	81	71	61	52	...	...	...
	9/16	117*	111	106	100	95	89	84	78	72	67	61	56	50	...	...	...	...	...	...	108	99	91	82	74	65	57	48	...	...	...
	1/2	105	100	95	90	85	80	75	70	65	60	54	49	44	...	...	...	...	...	...	97	90	82	74	67	59	52	44	...	...	...
	7/16	92	88	83	79	75	70	66	61	57	52	48	43	39	...	...	...	...	...	...	86	79	73	66	60	53	46	40	...	...	...
	3/8	80	76	72	68	64	60	56	52	49	45	41	37	33	...	...	...	...	...	...	74	69	63	57	51	46	40	34	...	...	...
	5/16	67	64	60	57	54	50	47	44	40	37	34	31	27	...	...	...	...	...	...	63	58	53	48	44	39	34	29	...	...	...
4×3	5/8	111	106	100	95	89	84	78	73	68	62	57	51	46	...	...	...	...	...	...	104	96	88	80	72	64	57	49	...	...	...
	9/16	101	96	91	86	82	77	72	67	62	57	52	47	42	...	...	...	...	...	...	95	88	80	73	66	59	52	45	...	...	...
	1/2	91	87	82	78	73	69	65	60	56	52	47	43	38	...	...	...	...	...	...	85	79	72	66	60	53	47	40	...	...	...
	7/16	80	76	73	69	65	61	57	53	49	46	42	38	34	...	...	...	...	...	...	75	70	64	59	53	48	42	36	...	...	...
	3/8	69*	66	63	60	56	53	50	46	43	40	36	33	30	...	...	...	...	...	...	65	60	56	51	46	41	37	32	27	...	...
	5/16	59*	56	53	50	48	45	42	39	36	34	31	28	25	...	...	...	...	...	...	55	51	47	43	39	35	31	27	23	...	...
3½×3	9/16	91	86	81	75	70	65	60	54	49	44	39	...	...	...	...	...	...	...	...	88	81	75	68	62	55	49	42	...	...	...
	1/2	82	77	72	68	63	58	54	49	44	39	35	...	...	...	...	...	...	...	...	79	73	67	62	56	50	44	39	33	...	...
	7/16	72	68	64	60	56	52	48	44	39	35	31	...	...	...	...	...	...	...	...	70	65	60	55	50	45	40	35	30	...	...
	3/8	63	59	56	52	49	45	42	38	35	31	28	...	...	...	...	...	...	...	...	61	56	52	48	44	39	35	31	26	...	...
	5/16	53	50	47	44	41	38	35	32	29	26	23	...	...	...	...	...	...	...	...	51	47	44	40	37	33	29	26	22	...	...
3½×2½	1/2	75	71	67	63	58	54	50	46	41	37	33	...	...	...	...	...	...	...	...	68	62	55	48	42	35	...	...	...	...	...
	7/16	67	63	59	55	52	48	44	40	37	33	29	...	...	...	...	...	...	...	...	60	55	49	43	38	32	...	...	...	...	...
	3/8	58	55	51	48	45	42	39	35	32	29	26	...	...	...	...	...	...	...	...	53	48	43	38	33	28	23	...	...	...	...
	5/16	49	46	43	41	38	35	32	30	27	24	22	...	...	...	...	...	...	...	...	45	41	36	32	28	24	20	...	...	...	...
	1/4	39	37	35	33	31	28	26	24	22	19	17	...	...	...	...	...	...	...	...	36	33	30	26	23	20	17	...	...	...	...
3×2½	7/16	59	55	51	47	42	38	34	30	26	...	...	...	...	...	...	...	...	...	...	55	50	45	40	35	30	25	...	...	...	...
	3/8	51	48	44	41	37	34	30	27	23	...	...	...	...	...	...	...	...	...	...	48	44	40	35	31	27	22	...	...	...	...
	5/16	43	40	37	34	32	29	26	23	20	...	...	...	...	...	...	...	...	...	...	41	37	33	30	26	22	19	...	...	...	...
	1/4	35	33	30	28	26	23	21	19	16	...	...	...	...	...	...	...	...	...	...	33	30	27	24	21	18	16	...	...	...	...
2½×2	3/8	39	36	33	29	26	23	19	...	...	...	...	...	...	...	...	...	...	...	...	36	32	27	23	18	...	...	...	...	...	...
	5/16	33	31	28	25	22	19	17	...	...	...	...	...	...	...	...	...	...	...	...	31	27	23	19	15	...	...	...	...	...	...
	1/4	27	25	23	20	18	16	13	...	...	...	...	...	...	...	...	...	...	...	...	25	22	19	16	13	...	...	...	...	...	...
	3/16	21	19	17	16	14	12	10	...	...	...	...	...	...	...	...	...	...	...	...	19	17	15	12	10	...	...	...	...	...	...

NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE
101	10201	151	22801	201	40401	251	63001	301	90601	351	123201	401	160801	451	203401	501	251001
102	10404	152	23104	202	40804	252	63504	302	91204	352	123904	402	161604	452	204304	502	252004
103	10609	153	23409	203	41209	253	64009	303	91809	353	124609	403	162409	453	205209	503	253009
104	10816	154	23716	204	41616	254	64516	304	92416	354	125316	404	163216	454	206116	504	254016
105	11025	155	24025	205	42025	255	65025	305	93025	355	126025	405	164025	455	207025	505	255025
106	11236	156	24336	206	42436	256	65536	306	93636	356	126736	406	164836	456	207936	506	256036
107	11449	157	24649	207	42849	257	66049	307	94249	357	127449	407	165649	457	208849	507	257049
108	11664	158	24964	208	43264	258	66564	308	94864	358	128164	408	166464	458	209764	508	258064
109	11881	159	25281	209	43681	259	67081	309	95481	359	128881	409	167281	459	210681	509	259081
110	12100	160	25600	210	44100	260	67600	310	96100	360	129600	410	168100	460	211600	510	260100
111	12321	161	25921	211	44521	261	68121	311	96721	361	130321	411	168921	461	212521	511	261121
112	12544	162	26244	212	44944	262	68644	312	97344	362	131044	412	169744	462	213444	512	262144
113	12769	163	26569	213	45369	263	69169	313	97969	363	131769	413	170569	463	214369	513	263169
114	12996	164	26896	214	45796	264	69696	314	98596	364	132496	414	171396	464	215296	514	264196
115	13225	165	27225	215	46225	265	70225	315	99225	365	133225	415	172225	465	216225	515	265225
116	13456	166	27556	216	46656	266	70756	316	99856	366	133956	416	173056	466	217156	516	266256
117	13689	167	27889	217	47089	267	71289	317	100489	367	134689	417	173889	467	218089	517	267289
118	13924	168	28224	218	47524	268	71824	318	101124	368	135424	418	174724	468	219024	518	268324
119	14161	169	28561	219	47961	269	72361	319	101761	369	136161	419	175561	469	219961	519	269361
120	14400	170	28900	220	48400	270	72900	320	102400	370	136900	420	176400	470	220900	520	270400
121	14641	171	29241	221	48841	271	73441	321	103041	371	137641	421	177241	471	221841	521	271441
122	14884	172	29584	222	49284	272	73984	322	103684	372	138384	422	178084	472	222784	522	272484
123	15129	173	29929	223	49729	273	74529	323	104329	373	139129	423	178929	473	223729	523	273529
124	15376	174	30276	224	50176	274	75076	324	104976	374	139876	424	179776	474	224676	524	274576
125	15625	175	30625	225	50625	275	75625	325	105625	375	140625	425	180625	475	225625	525	275625
126	15876	176	30976	226	51076	276	76176	326	106276	376	141376	426	181476	476	226576	526	276676
127	16129	177	31329	227	51529	277	76729	327	106929	377	142129	427	182329	477	227529	527	277729
128	16384	178	31684	228	51984	278	77284	328	107584	378	142884	428	183184	478	228484	528	278784
129	16641	179	32041	229	52441	279	77841	329	108241	379	143641	429	184041	479	229441	529	279841
130	16900	180	32400	230	52900	280	78400	330	108900	380	144400	430	184900	480	230400	530	280900
131	17161	181	32761	231	53361	281	78961	331	109561	381	145161	431	185761	481	231361	531	281961
132	17424	182	33124	232	53824	282	79524	332	110224	382	145924	432	186624	482	232324	532	283024
133	17689	183	33489	233	54289	283	80089	333	110889	383	146689	433	187489	483	233289	533	284089
134	17956	184	33856	234	54756	284	80656	334	111556	384	147456	434	188356	484	234256	534	285156
135	18225	185	34225	235	55225	285	81225	335	112225	385	148225	435	189225	485	235225	535	286225
136	18496	186	34596	236	55696	286	81796	336	112896	386	148996	436	190096	486	236196	536	287296
137	18769	187	34969	237	56169	287	82369	337	113569	387	149769	437	190969	487	237169	537	288369
138	19044	188	35344	238	56644	288	82944	338	114244	388	150544	438	191844	488	238144	538	289444
139	19321	189	35721	239	57121	289	83521	339	114921	389	151321	439	192721	489	239121	539	290521
140	19600	190	36100	240	57600	290	84100	340	115600	390	152100	440	193600	490	240100	540	291600
141	19881	191	36481	241	58081	291	84681	341	116281	391	152881	441	194481	491	241081	541	292681
142	20164	192	36864	242	58564	292	85264	342	116964	392	153664	442	195364	492	242064	542	293764
143	20449	193	37249	243	59049	293	85849	343	117649	393	154449	443	196249	493	243049	543	294849
144	20736	194	37636	244	59536	294	86436	344	118336	394	155236	444	197136	494	244036	544	295936
145	21025	195	38025	245	60025	295	87025	345	119025	395	156025	445	198025	495	245025	545	297025
146	21316	196	38416	246	60516	296	87616	346	119716	396	156816	446	198916	496	246016	546	298116
147	21609	197	38809	247	61009	297	88209	347	120409	397	157609	447	199809	497	247009	547	299209
148	21904	198	39204	248	61504	298	88804	348	121104	398	158404	448	200704	498	248004	548	300304
149	22201	199	39601	249	62001	299	89401	349	121801	399	159201	449	201601	499	249001	549	301401
150	22500	200	40000	250	62500	300	90000	350	122500	400	160000	450	202500	500	250000	550	302500

NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE	NO.	SQUARE
551	303601	601	361201	651	423801	701	491401	751	564001	801	641601	851	724201	901	811801	951	904401
552	304704	602	362404	652	425104	702	492804	752	565504	802	643204	852	725904	902	813604	952	906304
553	305809	603	363609	653	426409	703	494209	753	567009	803	644809	853	727609	903	815409	953	908209
554	306916	604	364816	654	427716	704	495616	754	568516	804	646416	854	729316	904	817216	954	910116
555	308025	605	366025	655	429025	705	497025	755	570025	805	648025	855	731025	905	819025	955	912025
556	309136	606	367236	656	430336	706	498436	756	571536	806	649636	856	732736	906	820836	956	913936
557	310249	607	368449	657	431649	707	499849	757	573049	807	651249	857	734449	907	822649	957	915849
558	311364	608	369664	658	432964	708	501264	758	574564	808	652864	858	736164	908	824464	958	917764
559	312481	609	370881	659	434281	709	502681	759	576081	809	654481	859	737881	909	826281	959	919681
560	313600	610	372100	660	435600	710	504100	760	577600	810	656100	860	739600	910	828100	960	921600
561	314721	611	373321	661	436921	711	505521	761	579121	811	657721	861	741321	911	829921	961	923521
562	315844	612	374544	662	438244	712	506944	762	580644	812	659344	862	743044	912	831744	962	925444
563	316969	613	375769	663	439569	713	508369	763	582169	813	660969	863	744769	913	833569	963	927369
564	318096	614	376996	664	440896	714	509796	764	583696	814	662596	864	746496	914	835396	964	929296
565	319225	615	378225	665	442225	715	511225	765	585225	815	664225	865	748225	915	837225	965	931225
566	320356	616	379456	666	443556	716	512656	766	586756	816	665856	866	749956	916	839056	966	933156
567	321489	617	380689	667	444889	717	514089	767	588289	817	667489	867	751689	917	840889	967	935089
568	322624	618	381924	668	446224	718	515524	768	589824	818	669124	868	753424	918	842724	968	937024
569	323761	619	383161	669	447561	719	516961	769	591361	819	670761	869	755161	919	844561	969	938961
570	324900	620	384400	670	448900	720	518400	770	592900	820	672400	870	756900	920	846400	970	940900
571	326041	621	385641	671	450241	721	519841	771	594441	821	674041	871	758641	921	848241	971	942841
572	327184	622	386884	672	451584	722	521284	772	595984	822	675684	872	760384	922	850084	972	944784
573	328329	623	388129	673	452929	723	522729	773	597529	823	677329	873	762129	923	851929	973	946729
574	329476	624	389376	674	454276	724	524176	774	599076	824	678976	874	763876	924	853776	974	948676
575	330625	625	390625	675	455625	725	525625	775	600625	825	680625	875	765625	925	855625	975	950625
576	331776	626	391876	676	456976	726	527076	776	602176	826	682276	876	767376	926	857476	976	952576
577	332929	627	393129	677	458329	727	528529	777	603729	827	683929	877	769129	927	859329	977	954529
578	334084	628	394384	678	459684	728	529984	778	605284	828	685584	878	770884	928	861184	978	956484
579	335241	629	395641	679	461041	729	531441	779	606841	829	687241	879	772641	929	863041	979	958441
580	336400	630	396900	680	462400	730	532900	780	608400	830	688900	880	774400	930	864900	980	960400
581	337561	631	398161	681	463761	731	534361	781	609961	831	690561	881	776161	931	866761	981	962361
582	338724	632	399424	682	465124	732	535824	782	611524	832	692224	882	777924	932	868624	982	964324
583	339889	633	400689	683	466489	733	537289	783	613089	833	693889	883	779689	933	870489	983	966289
584	341056	634	401956	684	467856	734	538756	784	614656	834	695556	884	781456	934	872356	984	968256
585	342225	635	403225	685	469225	735	540225	785	616225	835	697225	885	783225	935	874225	985	970225
586	343396	636	404496	686	470596	736	541696	786	617796	836	698896	886	784996	936	876096	986	972196
587	344569	637	405769	687	471969	737	543169	787	619369	837	700569	887	786769	937	877969	987	974169
588	345744	638	407044	688	473344	738	544644	788	620944	838	702244	888	788544	938	879844	988	976144
589	346921	639	408321	689	474721	739	546121	789	622521	839	703921	889	790321	939	881721	989	978121
590	348100	640	409600	690	476100	740	547600	790	624100	840	705600	890	792100	940	883600	990	980100
591	349281	641	410881	691	477481	741	549081	791	625681	841	707281	891	793881	941	885481	991	982081
592	350464	642	412164	692	478864	742	550564	792	627264	842	708964	892	795664	942	887364	992	984064
593	351649	643	413449	693	480249	743	552049	793	628849	843	710649	893	797449	943	889249	993	986049
594	352836	644	414736	694	481636	744	553536	794	630436	844	712336	894	799236	944	891136	994	988036
595	354025	645	416025	695	483025	745	555025	795	632025	845	714025	895	801025	945	893025	995	990025
596	355216	646	417316	696	484416	746	556516	796	633616	846	715716	896	802816	946	894916	996	992016
597	356409	647	418609	697	485809	747	558009	797	635209	847	717409	897	804609	947	896809	997	994009
598	357604	648	419904	698	487204	748	559504	798	636804	848	719104	898	806404	948	898704	998	996004
599	358801	649	421201	699	488601	749	561001	799	638401	849	720801	899	808201	949	900601	999	998001
600	360000	650	422500	700	490000	750	562500	800	640000	850	722500	900	810000	950	902500	1000	1000000

RESISTING MOMENTS OF PINS IN POUND-INCHES

DIAM.	AREA	UNIT STRESS FOR EXTREME FIBER IN POUNDS PER SQUARE INCH						
INS.	SQ. IN.	15,000	18,000	20,000	22,000	22,500	24,000	25,000
1½	1.77	4,970	5,960	6,630	7,290	7,460	7,950	8,280
1⅝	2.07	6,320	7,580	8,430	9,270	9,480	10,100	10,500
1¾	2.41	7,890	9,470	10,500	11,600	11,800	12,600	13,200
1⅞	2.76	9,710	11,600	12,900	14,200	14,600	15,500	16,200
2	3.14	11,800	14,100	15,700	17,300	17,700	18,900	19,600
2⅛	3.55	14,100	17,000	18,800	20,700	21,200	22,600	23,600
2¼	3.98	16,800	20,100	22,400	24,600	25,200	26,800	28,000
2⅜	4.43	19,700	23,700	26,300	28,900	29,600	31,600	32,900
2½	4.91	23,000	27,600	30,700	33,700	34,500	36,800	38,300
2⅝	5.41	26,600	32,000	35,500	39,100	40,000	42,600	44,400
2¾	5.94	30,600	36,700	40,800	44,900	45,900	49,000	51,000
2⅞	6.49	35,000	42,000	46,700	51,300	52,500	56,000	58,300
3	7.07	39,800	47,700	53,000	58,300	59,600	63,600	66,300
3⅛	7.67	44,900	53,900	59,900	65,900	67,400	71,900	74,900
3¼	8.30	50,600	60,700	67,400	74,100	75,800	80,900	84,300
3⅜	8.95	56,600	67,900	75,500	83,000	84,900	90,600	94,400
3½	9.62	63,100	75,800	84,200	92,600	94,700	101,000	105,200
3⅝	10.32	70,200	84,200	93,500	102,900	105,200	112,200	116,900
3¾	11.05	77,700	93,200	103,500	113,900	116,500	124,300	129,400
3⅞	11.79	85,700	102,800	114,200	125,700	128,500	137,100	142,800
4	12.57	94,200	113,100	125,700	138,200	141,400	150,800	157,100
4¼	14.19	113,000	135,700	150,700	165,800	169,600	180,900	188,400
4½	15.90	134,200	161,000	178,900	196,800	201,300	214,700	223,700
4¾	17.72	157,800	189,400	210,400	231,500	236,700	252,500	263,000
5	19.64	184,100	220,900	245,400	270,000	276,100	294,500	306,800
5¼	21.65	213,100	255,700	284,100	312,500	319.600	340,900	355,200
5½	23.76	245,000	294,000	326,700	359,300	367,500	392,000	408,300
5¾	25.97	280,000	336,000	373,300	410,600	419,900	447,900	466,600
6	28.27	318,100	381,700	424,100	466,500	477,100	508,900	530,100
6¼	30.68	359,500	431,400	479,400	527,300	539,300	575,200	599,200
6½	33.18	404,400	485,300	539,200	593,100	606,600	647,100	674,000
6¾	35.79	452,900	543,500	603,900	664,300	679,400	724,600	754,800
7	38.49	505,100	606,100	673,500	740,800	757,700	808,200	841,800
7¼	41.28	561,200	673,400	748,200	823,100	841,800	897,900	935,300
7½	44.18	621,300	745,500	828,400	911,200	931,900	994,000	1,035,000
7¾	47.17	685,500	822,600	914,000	1,005,000	1,028,000	1,097,000	1,142,000
8	50.27	754,000	904,800	1,005,000	1,106,000	1,131,000	1,206,000	1,257,000
8¼	53.46	826,900	992,300	1,103,000	1,213,000	1,240,000	1,323,000	1,378,000
8½	56.75	904,400	1,085,000	1,206,000	1,326,000	1,357,000	1,447,000	1,507,000
8¾	60.13	986,500	1,184,000	1,315,000	1,447,000	1,480,000	1,578,000	1,644,000
9	63.62	1,074,000	1.288,000	1,431,000	1,575,000	1,610,000	1,718,000	1,789,000
9¼	67.20	1,166,000	1,399,000	1,554,000	1,709,000	1,748,000	1,865,000	1,943,000
9½	70.88	1,263,000	1,515,000	1,683,000	1,852,000	1,894,000	2,020,000	2,104,000
9¾	74.66	1,365,000	1,638,000	1,820,000	2,002,000	2,047,000	2,184,000	2,275,000
10	78.54	1,473,000	1,767,000	1,963,000	2,160,000	2,209,000	2,356,000	2,454,000

DECIMAL EQUIVALENTS

DECIMALS OF A FOOT													DECIMALS OF AN INCH	
INS.	0	1	2	3	4	5	6	7	8	9	10	11		
0	.0000	.0833	.1667	.2500	.3333	.4167	.5000	.5833	.6667	.7500	.8333	.9167	....	....
1/32	.0026	.0859	.1693	.2526	.3359	.4193	.5026	.5859	.6693	.7526	.8359	.9193	1/32	.0313
1/16	.0052	.0885	.1719	.2552	.3385	.4219	.5052	.5885	.6719	.7552	.8385	.9219	1/16	.0625
3/32	.0078	.0911	.1745	.2578	.3411	.4245	.5078	.5911	.6745	.7578	.8411	.9245	3/32	.0938
⅛	.0104	.0938	.1771	.2604	.3438	.4271	.5104	.5938	.6771	.7604	.8438	.9271	⅛	.1250
5/32	.0130	.0964	.1797	.2630	.3464	.4297	.5130	.5964	.6797	.7630	.8464	.9297	5/32	.1563
3/16	.0156	.0990	.1823	.2656	.3490	.4323	.5156	.5990	.6823	.7656	.8490	.9323	3/16	.1875
7/32	.0182	.1016	.1849	.2682	.3516	.4349	.5182	.6016	.6849	.7682	.8516	.9349	7/32	.2188
¼	.0208	.1042	.1875	.2708	.3542	.4375	.5208	.6042	.6875	.7708	.8542	.9375	¼	.2500
9/32	.0234	.1068	.1901	.2734	.3568	.4401	.5234	.6068	.6901	.7734	.8568	.9401	9/32	.2813
5/16	.0260	.1094	.1927	.2760	.3594	.4427	.5260	.6094	.6927	.7760	.8594	.9427	5/16	.3125
11/32	.0286	.1120	.1953	.2786	.3620	.4453	.5286	.6120	.6953	.7786	.8620	.9453	11/32	.3438
⅜	.0313	.1146	.1979	.2813	.3646	.4479	.5313	.6146	.6979	.7813	.8646	.9479	⅜	.3750
13/32	.0339	.1172	.2005	.2839	.3672	.4505	.5339	.6172	.7005	.7839	.8672	.9505	13/32	.4063
7/16	.0365	.1198	.2031	.2865	.3698	.4531	.5365	.6198	.7031	.7865	.8698	.9531	7/16	.4375
15/32	.0391	.1224	.2057	2891	.3724	.4557	.5391	.6224	.7057	.7891	.8724	.9557	15/32	.4688
½	.0417	.1250	.2083	.2917	.3750	.4583	.5417	.6250	.7083	.7017	.8750	.9583	½	.5000
17/32	.0443	.1276	.2109	.2943	.3776	.4609	.5443	.6276	.7109	.7943	.8776	.9609	17/32	.5313
9/16	.0469	.1302	.2135	.2969	.3802	.4635	.5469	.6302	.7135	.7969	.8802	.9635	9/16	.5625
19/32	.0495	.1328	.2161	.2995	.3828	.4661	.5495	.6328	.7161	.7995	.8828	.9661	19/32	.5938
⅝	.0521	.1354	.2188	.3021	.3854	.4688	.5521	.6354	.7188	.8021	.8854	.9688	⅝	.6250
21/32	.0547	.1380	.2214	.3047	.3880	.4714	.5547	.6380	.7214	.8047	.8880	.9714	21/32	.6563
11/16	.0573	.1406	.2240	.3073	.3906	.4740	.5573	.6406	.7240	.8073	.8906	.9740	11/16	.6875
23/32	.0599	.1432	.2266	.3099	.3932	.4766	.5599	.6432	.7266	.8099	.8932	.9766	23/32	.7188
¾	.0625	.1458	.2292	.3125	.3958	.4792	.5625	.6458	.7292	.8125	.8958	.9792	¾	.7500
25/32	.0651	.1484	.2318	.3151	.3984	.4818	.5651	.6484	.7318	.8151	.8984	.9818	25/32	.7813
13/16	.0677	.1510	.2344	.3177	.4010	.4844	.5677	.6510	.7344	.8177	.9010	.9844	13/16	.8125
27/32	.0703	.1536	.2370	.3203	.4036	.4870	.5703	.6536	.7370	.8203	.9036	.9870	27/32	.8438
⅞	.0729	.1563	.2396	.3229	.4063	.4896	.5729	.6563	.7396	.8229	.9063	.9896	⅞	.8750
29/32	.0755	.1589	.2422	.3255	.4089	.4922	.5755	.6589	.7422	.8255	.9089	.9922	29/32	.9063
15/16	.0781	.1615	.2448	.3281	.4115	.4948	.5781	.6615	.7448	.8281	.9115	.9948	15/16	.9375
31/32	.0807	.1641	.2474	.3307	.4141	.4974	.5807	.6641	.7474	.8307	.9141	.9974	31/32	.9688

DESCRIPTION OF THE TABLES AND DIAGRAMS

PAGE 298

Weights and Dimensions of Carnegie I-beams. — Some of these beams rolled by the Carnegie Steel Company differ from those tabulated on the opposite page. The depth in inches is given under "Size," and the weight given is the weight per linear foot of beam. The weights of the preferred beams most commonly used are given in larger type. The heavier beams of each group are made by means of the same rolls, but the rolls are separated. The lighter supplementary beams, indicated by asterisks, are specially designed with wider flanges and thinner webs to give greater resistance to bending in proportion to their weights. The web thicknesses are given in both decimal and fractional forms. The flange widths and the web thicknesses are converted into fractions upon the basis noted at the top of the table. The grip t is the thickness of the flange at the rivet line located by the standard gage g. The tangent distance between the curved fillets may be found by subtracting twice the distance k from the depth. The standard length of bearing of a beam on a masonry wall is the first dimension (width) of the corresponding bearing plate found in the column headed p.

American Bridge Company Connection Angles. — The numbers of rivets in these connection angles differ from those in the angles tabulated on the opposite page, but they have been tested under ordinary conditions and found to be satisfactory. A constant distance of $5\frac{1}{2}''$ between the holes in the outstanding legs is used, and the corresponding gage b varies with the web thickness. A single 6×6 angle may be used on the lighter beams when two angles cannot be used. A single angle should not be used on the larger beams unless specially designed. The weights of the heads of the $\frac{3}{4}''$ shop rivets are included in the weights of the connection angles. The distance c, equal to one-half the web thickness plus $\frac{1}{16}''$, may be used to advantage in determining the length back to back of angles for a beam which is supported by an I-beam.

PAGE 299

Weights and Dimensions of Standard I-beams. — Similar to the tables described above.

Lackawanna Connection Angles. — These connection angles are similar to those described above. The numbers and the spacing of the rivets and the sizes of the angles differ. A gage of $2\frac{1}{4}''$ is used in the outstanding legs, and the corresponding distance a from center to center of holes varies with the web thickness. A variable gage b is given also for use with a constant dimension of $5\frac{1}{2}''$ if preferred.

PAGE 300

Weights and Dimensions of Carnegie Channels. — Similar to the table of Carnegie I-beams described above. Dimensions are given also for spacing the channels in channel columns. Note that the flange faces of the webs and the rivet lines are both kept a constant distance apart for a given depth of channel, but the gage varies with the web thickness.

American Bridge Company Connection Angles. — Similar to the table of American Bridge Company connection angles for I-beams described above. The holes are symmetrical about the center of the web, as for I-beams. There is an additional dimension c' equal to the full web thickness plus $\frac{1}{16}''$.

PAGE 301

Weights and Dimensions of Standard Channels. — Similar to the table of Carnegie channels described above.

Lackawanna Connection Angles. — Similar to the table of Lackawanna connection angles for I-beams described above. A distance h from the back of the channel to the holes in the angles is shown, and also a dimension c' equal to the full web thickness plus $\frac{1}{16}''$.

PAGE 302

Weights and Dimensions of Bethlehem I-beams and Girder Beams. — The left half of the table shows the special I-beams rolled by the Bethlehem Steel Company, and the right half the girder beams rolled by the same company. The dimensions are tabulated as in the preceding tables.

Bethlehem Connection Angles. — These connection angles are arranged as in the preceding tables.

PAGE 303

Weights and Areas of Angles. — The weights per linear foot and the areas of cross section in square inches are tabulated for standard angles at the left and for special angles at the right. This arrangement simplifies the selection of a standard angle to fulfill requirements.

Gages of Angles. — The standard gages commonly used for different sizes of angles.

Areas of Rivet Holes. — This table gives the areas of cross section (rectangles) of holes of different diameters in metal of different thicknesses. Note that the diameter of the hole is taken 1″ greater than the nominal diameter of the corresponding rivet.

PAGE 304

Weights and Dimensions of Rivets and Bolts. — These tables are self-explanatory.

Rivet Code. — The inner rows show the conventional method of indicating rivets with different forms of heads. The outer rows represent the corresponding rivets.

Clearance for Machine-driven Rivets. — This shows the desired distance and the minimum distance from the center of a rivet to any projecting part which might interfere with the use of a riveting machine.

Minimum Rivet Stagger. — This shows the minimum stagger of rivets, in opposite legs of an angle, which will provide the driving clearance tabulated at the left of the page.

PAGE 305

Minimum Rivet Spacing. — This shows the minimum spaces used for ordinary work. The minimum pitches in the flanges of plate girders are shown in the table on the following page.

Maximum Rivet Spacing. — This shows the maximum spaces under ordinary conditions.

Edge Distance. — This shows the minimum and maximum values for the perpendicular distance from the center of a rivet to the sheared edge of any piece. The distance to a rolled edge may be less necessarily.

Minimum Rivet Stagger. — There are two sets of curves for determining the minimum stagger of rivets. The circular curves show when the diagonal distance from center to center of holes equals the usual minimum given in the table at the top of the page. The other curves show how close together two rivets may be placed in a tension member without the necessity of considering both in finding the limiting net cross section, as explained more fully on page 209 : 1. This diagram can be used even when the holes are in different legs of an angle, as indicated below the diagram. For the minimum stagger for the rivets in the flanges of plate girders, see the following table.

PAGE 306

Minimum Pitches for Flange Rivets. — These tables give the minimum pitches for the rivets in the flanges of plate girders under different conditions. Their importance and their use are described more fully on page 255 : 2. The upper table should be used to conform to the specifications of the American Railway Engineering Association.

PAGE 307

Multiplication Table for Rivet Spacing. — This table is self-explanatory. Values for pitches in sixteenths can be interpolated or found by taking one-half the values which correspond to pitches twice as large.

PAGES 308 TO 311

Rivet Values. — These four tables are practically self-explanatory, but they are described on page 231 : 1.

PAGE 312

Graphic Resultants — Decimals. This diagram may be used to advantage in finding the resultants of forces or stresses in pounds or in thousands of pounds. One component is measured horizontally, the other vertically, while the resultant is measured diagonally. There are five spaces between numbers, the odd tenths falling in the centers of the spaces. The diagram can often be used to better advantage if both components are multiplied or divided by a convenient factor, and the resultant corrected to correspond.

PAGE 313

Graphic Resultants — Inches and Fractions. This diagram may be used to advantage in finding small diagonal distances in inches and fractions from rectangular coördinates, in much the same manner as in the preceding diagram. There are eight spaces between numbers, the sixteenths being interpolated.

PAGE 314

Graphic Resultants — Feet and Inches. This diagram may be used in finding diagonal distances in feet and inches from rectangular coördinates, in the same manner as in the two preceding diagrams. There are six spaces between numbers, the inches being interpolated. This diagram cannot be expected to replace a table of squares in computing lengths of diagonals to the nearest sixteenths of an inch, but it can be used for less precise work.

PAGE 315

Purlin Connections. — Typical standard purlin connections to roof trusses or rafters are here shown in detail. The angles are usually $\frac{5}{16}''$ thick.

Lattice Bars. — The maximum lengths of lattice bars for different thicknesses are shown for different specifications. The values given in the left-hand columns for both single and double latticing correspond to the more usual specifications. The detailed dimensions at the ends of lattice bars are shown also.

Areas and Weights of Rods. — The gross areas of round rods, and the diameters and the lengths of standard upset ends are given. The areas at the roots of the threads of the upset ends are shown. These root areas may be used also for rods which are not upset, intermediate values being tabulated on page 207:4.

PAGE 316

Bearing Plates. — The relation between the thickness of a bearing plate and its projection beyond the superimposed metal is shown in the diagram. The unit stress in bending in the extreme fiber of the steel plate is 16,000#/sq. in., but different lines indicate different bearing values allowed on the masonry. If the projection is known, the required thickness may be determined by the vertical distance to the intersection of the proper vertical and diagonal lines. Conversely, the maximum projection of a plate of a given thickness may be found.

Separators. — Detailed dimensions of cast-iron separators for I-beam girders of different sizes are shown, and the weights of the separators and the bolts used with them are given.

Anchors. — Details are shown of Government anchors and angle anchors for beams, and of swedge bolts and built-in anchor bolts for other classes of work.

Rod Connections. — Different methods are indicated for attaching rods to other members. Details are given for the ends of tie rods or sag rods.

PAGE 317

Dimensions and Properties of Rails. — Detailed dimensions, areas, and section moduli of rails of different weights per *yard* are shown for the different standards (page 44:7).

Rail Fastenings. — Details are given for the splice bars for crane rails, with punching to match the standard mill punching at the ends of the rails. Hook bolts for fastening crane rails to the tops of I-beams, rail clamps for fastening crane rails to the tops of plate girders, and crane stops to be bolted to the ends of crane rails are illustrated.

Unit Stresses for Structural Steel. — These are self-explanatory.

PAGE 318

Shear and Moment Table for Cooper's Engine Loading. — This table is largely self-explanatory, but its use is more fully described on page 193:1.

PAGE 319

Properties of Wooden Rectangular Beams. — This table shows the section moduli for rectangular beams, not only for nominal dimensions in even inches, but also for dimensions which conform to the actual sizes of sawed and planed lumber. The designer should be familiar with the practice of the lumber mills which furnish the wooden beams for any structure which he designs, and he should use the sizes which are likely to be delivered. The table shows also the area of cross section, the number of feet-board measure per foot of length, and the weight per linear foot for each section.

PAGE 320

Unit Stresses for Structural Timber. — The unit stresses are given for different kinds of structural timber, and for different types of structure, as clearly indicated in the table.

Moments of Inertia of Rectangles. — These moments of inertia are for rectangles in which the longer dimension is at right angles to the neutral axis through the center. Moments of inertia of rectangles wider than 1 inch can be found by multiplying the values in the last column by the widths in inches.

PAGE 321

Areas of Plates. — The areas of plates most likely to be used are included in the table; others may be found by interpolation or by proportion. Note the inset portion which shows the areas of the wider plates. The net areas of cover plates or similar plates may be found from the table by using the net widths of the plates. These are found by deducting from the full widths the diameters of the rivet holes cut by any one section.

Weights of Plates. — The weights per linear foot are given for steel plates of different cross section. Note the inset portion for the weights of the wider plates.

PAGE 322

Properties of I-beams. — This table is self-explanatory. The I-beams are placed in two groups to correspond to those on pages 298 and 299. The web thicknesses are converted to fractions upon a different basis from that used in the first five tables, some of resulting values used in designing being $\frac{1}{16}''$ less than the corresponding values used in drafting.

PAGE 323

Properties of Channels. — This table is self-explanatory. See note regarding web thicknesses in the preceding paragraph.

PAGE 324

Properties of Bethlehem I-beams and Girder Beams. — These tables are self-explanatory. See note regarding web thickness under "Properties of I-beams" above.

List of Shipping Marks. — If further explanation is necessary, see Chapter XVI, page 79.

PAGES 325 AND 326

Properties of Standard and Special Angles. — For convenience, the properties of standard angles and special angles are placed on different pages. I_S, s_S, and r_S represent respectively the moment of inertia, the section modulus, and the radius of gyration about the axis through the center of gravity parallel to the shorter (or equal) leg, and x is the distance from the back of this leg to this axis. I_L, s_L and r_L represent the moment of inertia, the section modulus, and the radius of gyration about the axis through the center of gravity parallel to the longer leg, and y is the distance from the back of this leg to this axis. r_M is the minimum or least radius of gyration which is about a diagonal axis.

PAGE 327

Two Angles in Tension. — This table may be used to advantage in designing plate girders as well as two-angle tension members. The tensile stresses are used when the unit stress is 16,000#/sq. in. The net areas are used for other unit stresses.

PAGE 328

Unit Stresses for Compression. — This table gives the resulting unit stresses found by substituting different values for the length and the radius of gyration in the common column formula printed at the top of the table. A maximum of 14,000 is usually specified, and if so, it should be used in place of the values below the dotted lines in the lower left-hand portion of the table. The maximum ratio of slenderness for the main-compression members of bridges is usually specified as 100, while for buildings 125 is allowed. The maximum for secondary compression members is usually 120 for bridges and 150 for buildings. Zigzag lines are used to indicate these ratios.

PAGE 329

Radii of Gyration of Two Angle Struts. — The radii of gyration are given about axes parallel to both legs through the center of gravity of compression members composed of two angles spaced at different dis-

tances apart. This table can be used to advantage in designing compression members for other unit stresses from that used in the following tables.

PAGES 330 AND 331

Safe loads of Single Angle and Two Angle Struts. — These tables show the maximum total loads which can be placed upon compression members composed of one or two angles of different lengths without exceeding the unit stress shown at the tops of the tables. The zigzag lines indicate the limiting ratios for different types of members, as described above.

PAGE 332

Table of Squares. — This table is self-explanatory. The square of a number less than 100 can be found from the table by moving the decimal point of the number one place and that of the corresponding square two places. Similarly, the squares of numbers over 1000 can be found.

PAGE 333

Resisting Moments of Pins. — This table gives the resisting moments of cylindrical pins for different unit stresses.

Decimal Equivalents. — This table may be used to convert dimensions in inches and fractions of inches to the corresponding decimals of a foot, and conversely. It may also be used to convert fractions of an inch to decimals of an inch, and vice versa.

INDEX

(For definitions of engineering terms, see pages 6 to 18. For description of tables and diagrams, see pages 334 to 338.)
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